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Santa Clara University Multicultural Center Redesign

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SANTA CLARA UNIVERSITY

Department of Civil Engineering

I HEREBY RECOMMEND THAT THE THESIS PREPARED
UNDER MY SUPERVISION BY

Angela Non
Isaac Raven

ENTITLED

SANTA CLARA UNIVERSITY
MULTICULTURAL CENTER REDESIGN

BE ACCEPTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR THE DEGREE OF

BACHELOR OF SCIENCE
IN
CIVIL ENGINEERING

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2015

ABSTRACT

The purpose of this project is to propose a resigned structure to better suit the activities of the Santa Clara University Multicultural Center. The renovation entails proposing a preliminary structural design system that includes a new, light-weight PLN3 metal deck roofing system provided by Verco Decking, Inc. Nine deep long span 56DLH truss joists will be implemented to support the metal deck. Two large trusses comprised of member sizes HSS9x9x$\frac{1}{2}$ and LL3$\frac{1}{2}$x3$\frac{1}{2}$x$\frac{5}{16}$x$\frac{3}{8}$ will be used to support the nine truss joists. Four concrete columns, 15 ft in height and 2 ft in diameter, will be erected to uphold each end of the two steel trusses. Lastly, column footings of 4.5'x4.5'x3.0' will be used to support each concrete column. The architectural components of the renovation aim to include a complete redesign of the existing common area of the original building as well as implementing an extension to allow for more space. The architectural components include a new general meeting area layout, four private study rooms, one conference room, a desk reception and storage facility area, as well as an emergency exit extension and multipurpose activity center. The total cost of renovation is estimated to be $864,341.00 with a cost per square foot of $156.58. The duration of this project is expected to last 11 weeks, starting from June 15 to August 28.

ACKNOWLEDGEMENTS

First and foremost, we would like thank the Santa Clara University School of Engineering for allowing us to present this thesis and for giving us the skills and knowledge to do so. We would also like to acknowledge and thank Don Ackerland, Director of Planning and Projects at Santa Clara University, for leading us to resources on campus which included existing site plans and geotechnical reports. And lastly, we would like to thank Dr. Hisham Said and Professor Tracy Abbott for their outstanding support and guidance throughout this process.

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1 Introduction

1.1 Project Goals

The purpose of this project is to propose a structure that will better suit and accommodate the activities of Santa Clara University's multicultural clubs for the benefit of on-campus student life. The scope of this project entails providing a renovation plan for the Bob Shapell Student Activities Hall, home to the Multicultural Center on campus, that includes a preliminary schematic structural design of a new steel-truss-supported roof system and a remodeled architectural interior layout. In addition, this proposal will also delve into a construction management plan, including a preliminary Work Breakdown Structure (WBS), cost estimate, construction schedule, and site logistics plan to be used if and when this proposal is put into action.

1.2 Contributions

Given her interest in design as well as her experience in construction management, Angela Non has acted as Lead Structural Engineer and Project Manager for this project, and Isaac Raven, whose interest lies in architecture, will act as the Lead Architect and Building Information Modeling Designer. This proposal provides the Santa Clara community a new and renovated space for those that are involved in the Multicultural Center and its club activities as well as those who are interested in using the building's attributes.

1.3 Current Conditions

The Bob Shapell Student Activities Hall, circled in red in Figure 1 below, is located in Santa Clara University's Benson Plaza. This plaza is located in a central portion of the

SCU campus and is a main hub of student traffic given its proximity to the University Library, Graham Residence Hall, and Kenna Lecture Hall.

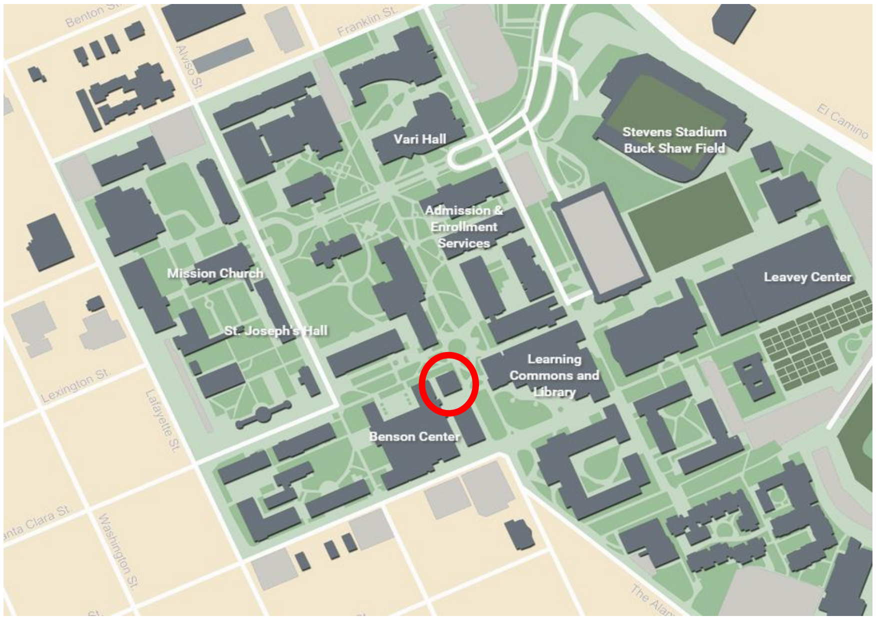

Figure 1. A campus map of Santa Clara University, indicating the location of the Bob Shapell Student Activities Hall.

The building of focus is located on the right-hand side of the Robert F. Benson Memorial Center and directly across the SCU Campus Bookstore, as illustrated in the current live view of the plaza below:

Figure 2. A view of the Bob Shapell Student Activities Hall (right) in relation to the Robert F. Benson Memorial Center (center) and SCU Campus Bookstore (left).

1.4 A Brief History of the Space

The Bob Shapell Student Activities Hall was added to the University as part of the 1983 proposal calling for the redesign of the Benson Memorial Center, which, in addition to the Hall, included the construction of the Campus Bookstore. Completed in 1985, this space was originally used as a recreational lounge for commuter students. In 2000, this commuter lounge was converted into the permanent home of Santa Clara University's Multicultural Center, also known as the MCC. For consistency, this building of focus will now be referred to as the MCC for the duration of this proposal. However, the history of the space, although brief, clearly indicates that the original layout and functions of the building were designed to meet outdated needs and no longer pertain to the current use of the space.

1.5 Current Use of Space

The MCC is a multicultural programming body that represents the racial and ethnic advocacy voice for the Santa Clara University community. This organization also overlooks and supports ten cultural student clubs as follows:

- Asian Pacific-Islander Student Union (APSU)
- Barkada (Filipino)
- Chinese Student Association (CSA)
- Igwebuike (Black/Pan-African)
- Intandesh (South Asian)
- Japanese Student Association (JSA)
- Ka Mana'o O Hawai'i
- Korean Student Association (KSA)
- MEChA-El Frente (Latina/o)
- Vietnamese Student Association (VSA)

Collectively, these ten cultural clubs are comprised of over 600 student members, thus accounting for roughly 11% of the total undergraduate student population that are affected by the amenities provided by the MCC.

The current layout of the MCC is split into two rooms. Upon entering the space, the larger room on the left-hand side provides a multi-purpose venue for the activities of the ten clubs. Each week, this space is used to house a minimum of ten general club meetings, with each meeting usually accommodating anywhere from 30 to 100 members in attendance. In addition, the MCC also holds educational seminars, panels, and forums that elaborate on issues that relate to the preservation of an environment conducive to the unique expression and appreciation of the various cultures of people of color. Lastly, when not in use for scheduled meetings or events, the MCC also provides a recreational and academic space for students to utilize.

The right-hand side of the MCC is a smaller subsection used as a combined Multicultural Center office and conference space. The office is used as a dedicated workspace to fulfill the administrative requirements of the MCC to maintain its status as a Registered Student Organization (RSO) recognized by the University. The conference space is needed to accommodate the eleven smaller executive board meetings held each week to plan the future and success of each cultural club. In addition, storage closets are also located along the back walls of this subsection to house the props and possessions of the MCC. Although cramped in space, most students tend to flock to this area as an unofficial academic workspace, which is only one of the many examples of how the MCC provides a dysfunctional and obscure layout for its users, one of the main concerns addressed in the proposed renovation.

1.6 Demonstration of Need

1.6.1 Dysfunctional Layout

As mentioned above, the dysfunctionality of the space is one of the main concerns that led to the call of a renovation. The pictures below indicate the current state of both rooms in the MCC, illustrating a cluttered, clustered, and disorganized layout.

Figure 3. A view of the general meeting space inside the MCC.

Figure 3 provides a look inside the general meeting area located on the left-hand side of the MCC. It is apparent that the room is not only disorganized and unclear as to which designated areas should be used for which functions, but the two concrete columns located in the middle of the space also interfere with the functionality and general flow of a space intended for large meetings. In short, the space intended for multi-purpose activities is cluttered, unorganized, and dysfunctional.

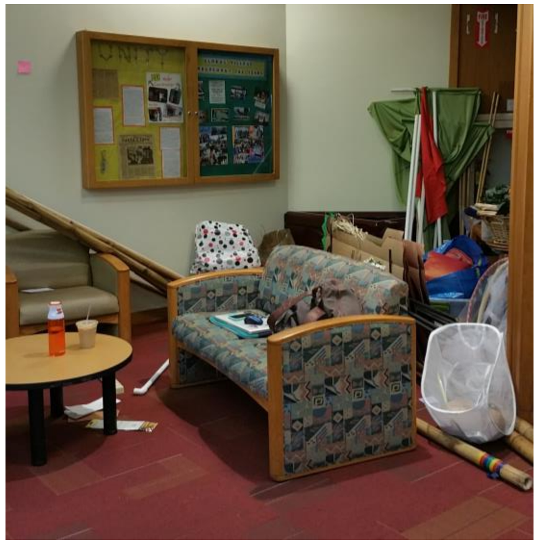

Figure 4. Another view of the general meeting area, highlighting the lack of storage, as shown through the cluttered material.

Figure 4 above demonstrates the need for more storage space to be incorporated in a renovation of the MCC. Mmany props, materials, and cultural dance items are left in the corners of the MCC as no other storage closets could be used to house them.

Figure 5. A view of the combined Multicultural Center office and conference room.

The MCC subsection, shown in Figure 5, is a cramped space that does not provide a sufficient enough area to allow its members to work productively.

1.6.2 *Voices of the MCC*

In order to further assess the need for a renovation to the space, a survey was conducted on 100 randomly selected members of the MCC regarding the suitability of the space. When asked if the amenities of the current MCC layout specifically catered to the needs of their organization, an overwhelming 88% of the participants disagreed, claiming that the features of the space no longer contributed to its current uses. From the results, it can be concluded that the need for an updated layout of the space is a popular opinion and should be addressed.

To better understand how to make the space more functional, the participants were then prompted to provide feedback regarding what exact features the space was lacking. They were given a list of five proposed features and asked to pick up to 3 that they would like to see included in a renovation of the of the MCC. The five items were as follows: more storage space, a redesign of the MCC office room/conference space, a more functional layout, rehearsal space for cultural dances, and an addition of more conference rooms/study space. The results of the survey indicated that a more functional layout, a rehearsal space, and smaller conference rooms were the most requested features, and thus, these elements were prioritized in the renovation plan of the MCC. Summaries of the survey results are illustrated in the graphs and charts below.

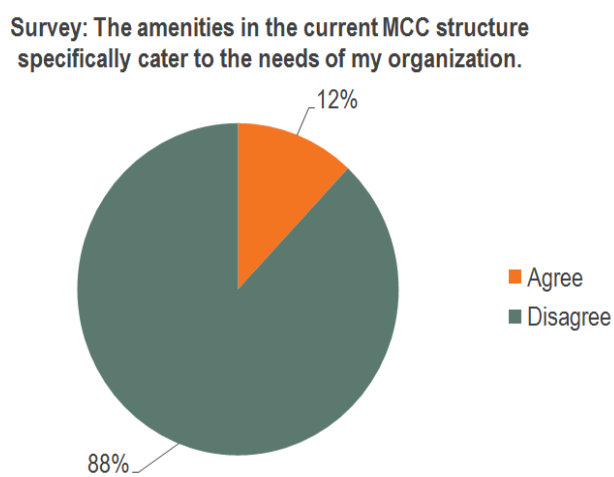

Figure 7. A survey conducted on 100 randomly selected students of the MCC regarding the usability of the MCC.

As illustrated from the graph above, a majority of the members of the MCC recognize that the amenities of the MCC no longer pertain to the current needs of their organization. The table below illustrates what features the MCC needs in a renovation to make the space more relevant to its current uses.

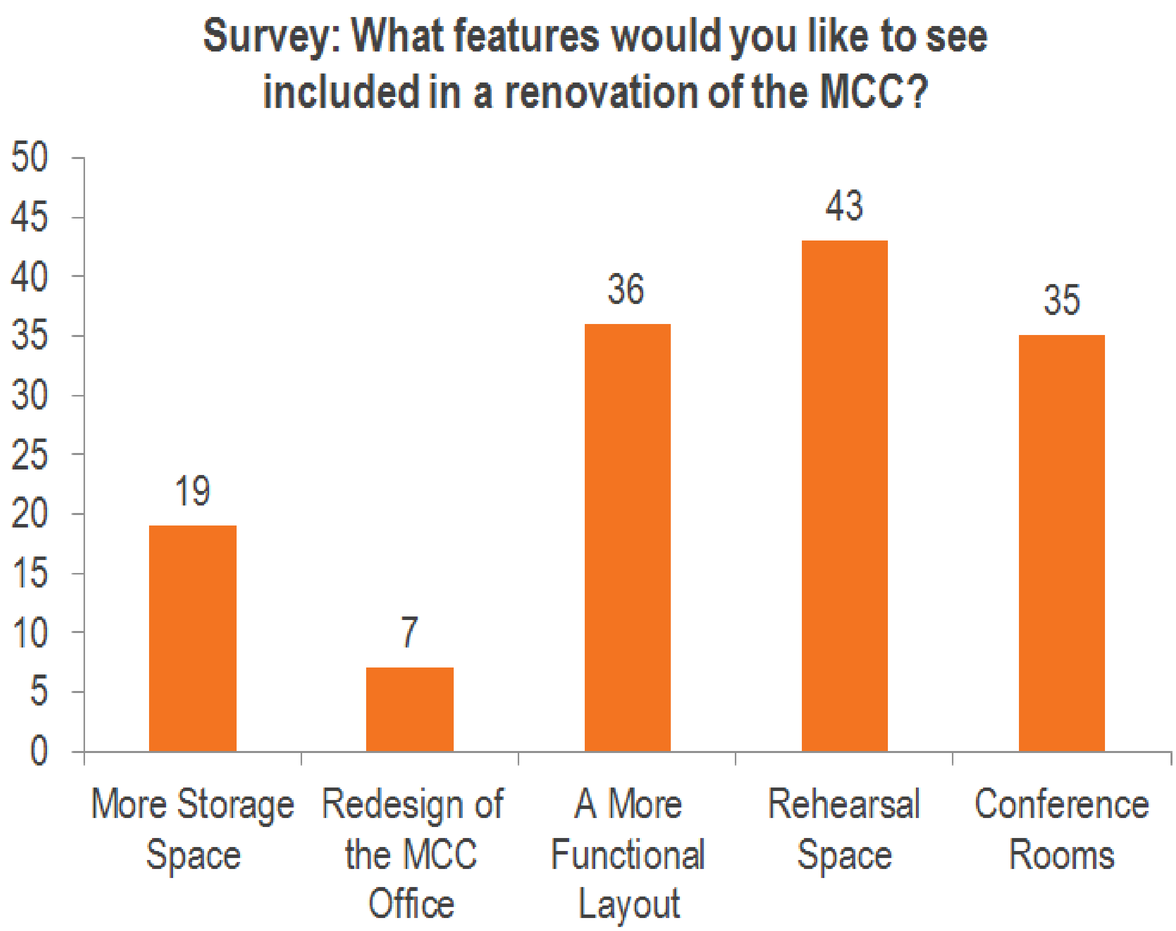

Figure 8. A summary of the features requested by members of the MCC to be included in the renovation of the MCC.

Figure 8 above indicates that the most requested features to be implemented into a renovation of the MCC includes a cultural dance rehearsal space for annual culture shows, a more functional design indicating designated areas for specific activities, and the presence of more conference rooms and study spaces.   Thus, the renovation will aim to incorporate the requests of the members of the MCC as presented in the survey above.

2 Architectural Elements

2.1 Current Conditions

For the architectural design of our proposed project, the main objective was to address the student needs as discussed earlier in the assessment of need of the project. The current architectural layout does not foster an environment that is conducive to learning and the various activities that take place in the space. The current building is split into two large rooms. One room contains a general meeting space where culture clubs and students hold their club meetings and activities. The second room contains a front desk reception and conference table alongside storage units placed in a small corner of the room. This space is especially tightly packed and is difficult to designate which part of the room will be for what purpose. Overall, the project is specifically designed to address the dysfunctionality and limited space of the current building and provide a more fluid, dynamic, and functional layout.

2.2 Proposed Interior Layout

The elements of the project redesign consist of a renovation of the interior layout and addition of an extension towards the north end of the building. The elements for the interior renovation include redesigning the general meeting space and improving additional lounge spaces, conference and study rooms, and an auxiliary space that contains storage and a front desk reception. The extension will be used to house multipurpose room and an emergency exit continuation. Figure 9 below is the proposed interior layout floorplan that demonstrates the placement of these architectural elements.

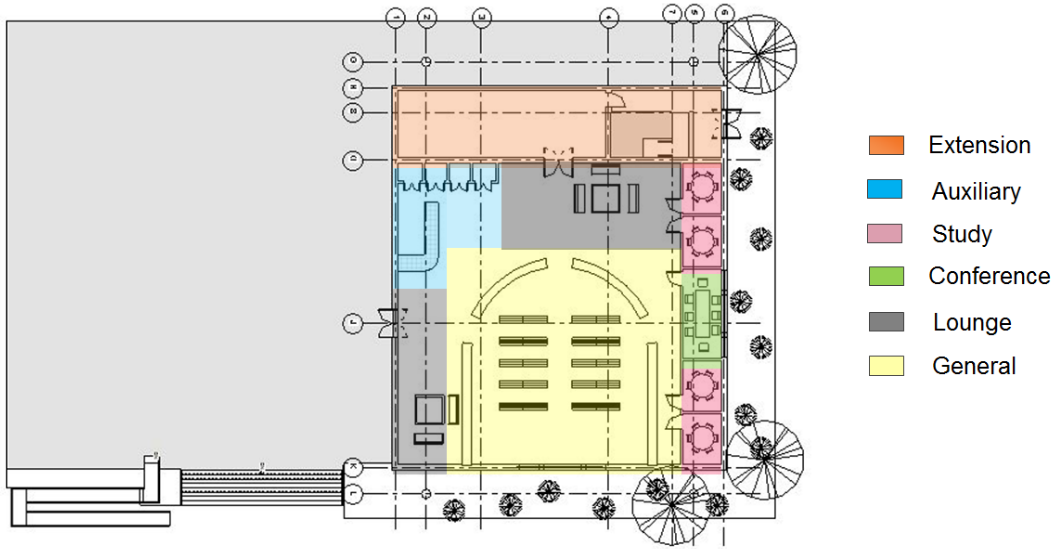

Figure 9. Proposed interior layout floorplan

2.2.1 General Meeting Space

One of the most important factors for the redesign was to provide a more conducive general meeting space. To complete this, an open atmosphere environment was the best choice to implement in the architectural design. As a part of our new design, 20 benches were installed encompassed by a counter partition to create a section specifically designed for general club and student organization meetings. Below are images taken from the Revit 3D architectural modeling software as well as the floor plan of this specific space.

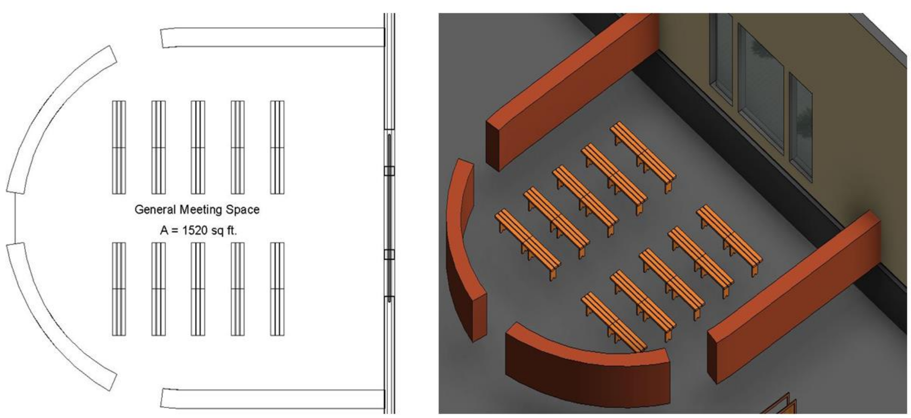

Figure 10. Proposed general meeting space in interior layout.

The general meeting space includes a partially closed off area of 1520 square feet towards the east end of the interior of the building.

2.2.2 Group Study Rooms and Conference Room

The next element as part of the redesign for the MCC was incorporating a private conference room as well as individual study spaces for students to utilize. From the student survey, the study/conference rooms were the third most requested feature from the participants. The current MCC does not have individual spaces sectioned off for studying; instead, it currently has scattered tables spread across the general meeting area with no sense of cohesiveness. Often, when there are club meetings going on in the general meeting space, those that are not involved are disturbed by the surrounding noise from the other activities that are taking place in the same room. The renovation aims to create spaces for students to study without being disturbed by extraneous noise from other MCC activities. Thus, the proposed interior layout includes four study rooms located on the east wing of the structure. These study spaces are each 9 ft by 11 ft and can house of to six students at the time. The conference room was positioned in center of the eastern wall with two study rooms on either side. This space is larger than the study rooms with a length of 19 feet and a width of 9 feet and can hold up to 8 people. All 5 rooms were created with sound proof doors and walls to create an environment that was suitable for important meetings or study groups that excluded static noise from the outside. Below is a floor plan that demonstrates the placement of the study and private conference areas.

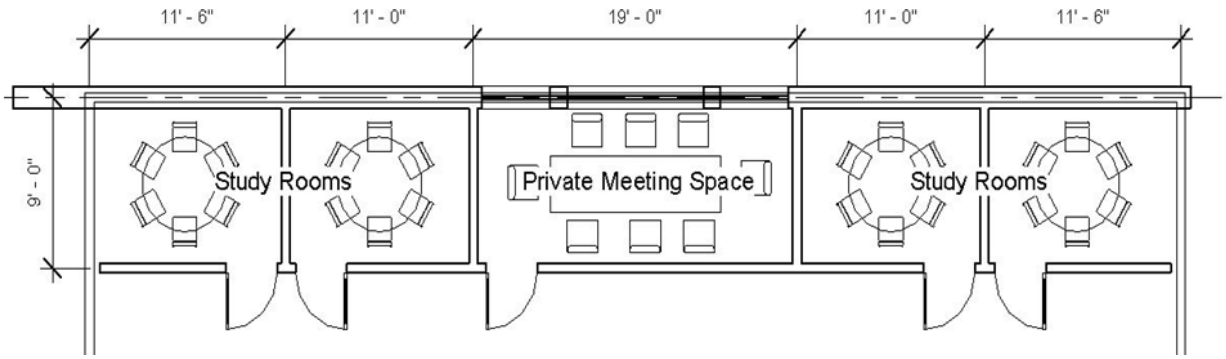

Figure 11. Proposed subsections of study rooms and private conference room in interior layout.

2.2.3 Auxiliary Space

The auxiliary space of the redesign contains storage facilities and a front desk reception area. Again, by addressing the needs of the students, part of the redesign was to include more storage per square foot for the entire building to encourage an organized layout and to provide an area for culture clubs and other students to place their general items. The front desk will be adjacent to the main entrance of the building to initiate a more welcoming presence for students who walk into the building. The front desk is indicated by a counter partition with a width of 3.5' and a total length of 16'-1" that wraps around in a 90 degree angle, as pictured in Figure 12.

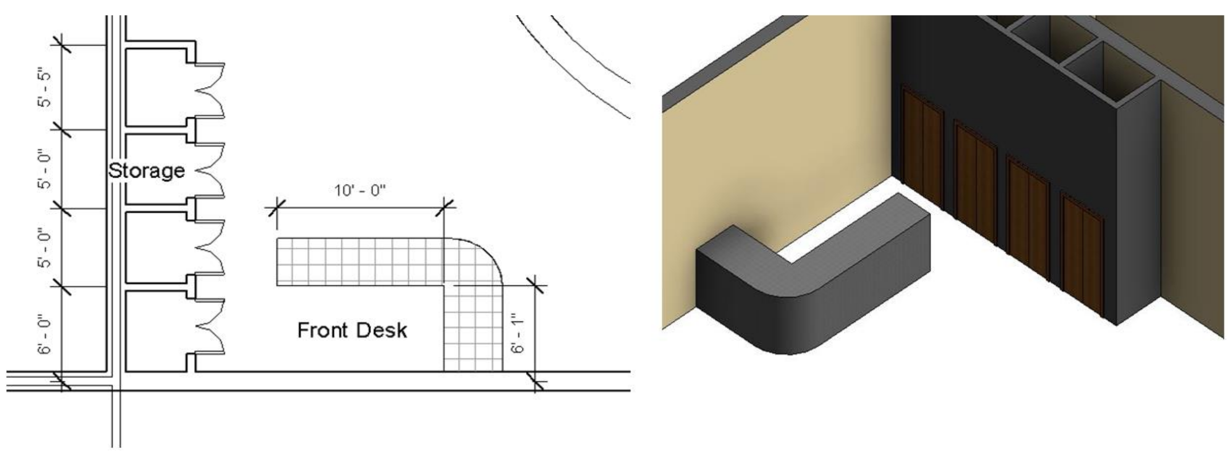

Figure 12. Proposed auxiliary space in interior layout. Includes front desk reception and storage facilities.

2.3 Extension

The second part of the architectural renovation for the MCC in our design project was to include an extension that housed both a multipurpose activity room and an emergency exit continuation. The location of this extension is shown in Figure 13 below.

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Figure 13. Proposed floor plan extension of MCC with respect to surrounding existing buildings.

The extension is made of nonstructural walls and the metal deck extends over this section, giving the entire proposed building a length of 80 feet and a continued width of 69 feet. This expansion will extend into what is currently used as a small, unused patio area. More often than not, this area on campus is neglected and usually empty. The current patio contains some seating and tables, as well as a counter partition that contains the emergency exit stairwell from the basement (Drahmann Center), as illustrated in Figure 14 below.

Figure 14. Current use of space between MCC and Benson Memorial Center

As illustrated above, the patio is hardly ever occupied by students and can be used for a more functional purpose. Therefore, in order to enhance the usability of this space, it can easily be converted into an extension of the building that will address the needs of the MCC by converting this space into a dual multipurpose activity room as well as an emergency exit continuation.

2.3.1 Multipurpose Activity Room

According to the survey taken from the MCC students, one of the highest requested features to be included in a renovation of the MCC was an addition of a multipurpose room designated specifically for a dance rehearsal space for the 5 annual cultural showcases the MCC holds throughout the year. The extension space will thus be converted into a dance studio for students to utilize.

The Multipurpose room is connected with the original doors of the MCC to the center of the room, and has a connection to the emergency exit continuation to allow those to be able to leave the room in case of emergency situations. The dimensions of this room are 15' x 45' for a total area of 671 feet squared. Below is a 3D isometric model of the entire proposed extension of the design project.

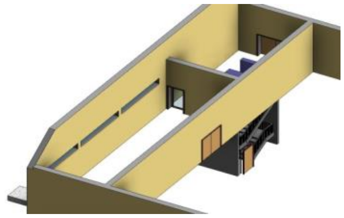


Figure 15. Proposed plans for multipurpose activity room, one element that pertains to the extension.

2.3.2 Emergency Exit Continuation

The emergency exit continuation was a critical feature of the original structure that needed to be preserved throughout the renovation, as it is used to provide an alternate exit for the Drahmann Tutoring Center in the basement below. Thus, in order to conform to codes requiring the presence of multiple exits, the emergency exit located in the current patio area will be maintained throughout the renovation.

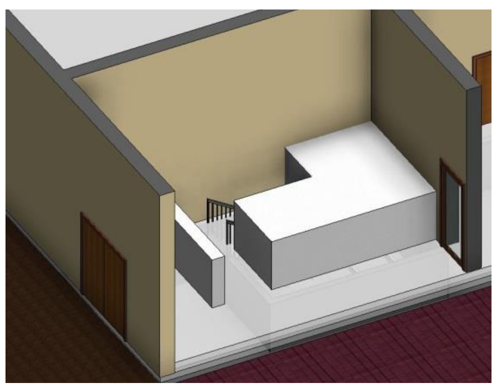


Figure 16. Emergency Exit continuation 3D model view for proposed extension.

The emergency exit continuation room has two doors, with one leading to the outside of the north face of the building and one the other connecting to the multipurpose room. This allows both the residents of the multipurpose room and Drahmann Center to exit the building safely. The emergency exit continuation is 15’ x 24.25’ for a total area of 364 square feet.

3 Structural Design

3.1 Original Constraints

As presented through drawings and plans provided by the Facilities Department of Santa Clara University, the current MCC is located in the top floor of a two-story cast-in-place concrete building, with the SCU Drahmann Tutoring Center on the first floor located underground. In addition, positioned in the center of building are four concrete columns that create the four corners of a square. These columns span the full height of the building through both stories and are used to hold up the cast-in-place floor of the top level as well as the cast-in-place concrete roof. Although we aim to renovate the top story of the building, it should be noted that the basement will be excluded from the renovation and that all necessary actions should be done in order to eliminate any disturbance to the bottom floor. A comparison of both the top live view of the Benson plaza and the framing elevation plans of the plaza can be found in Figure 17 below:

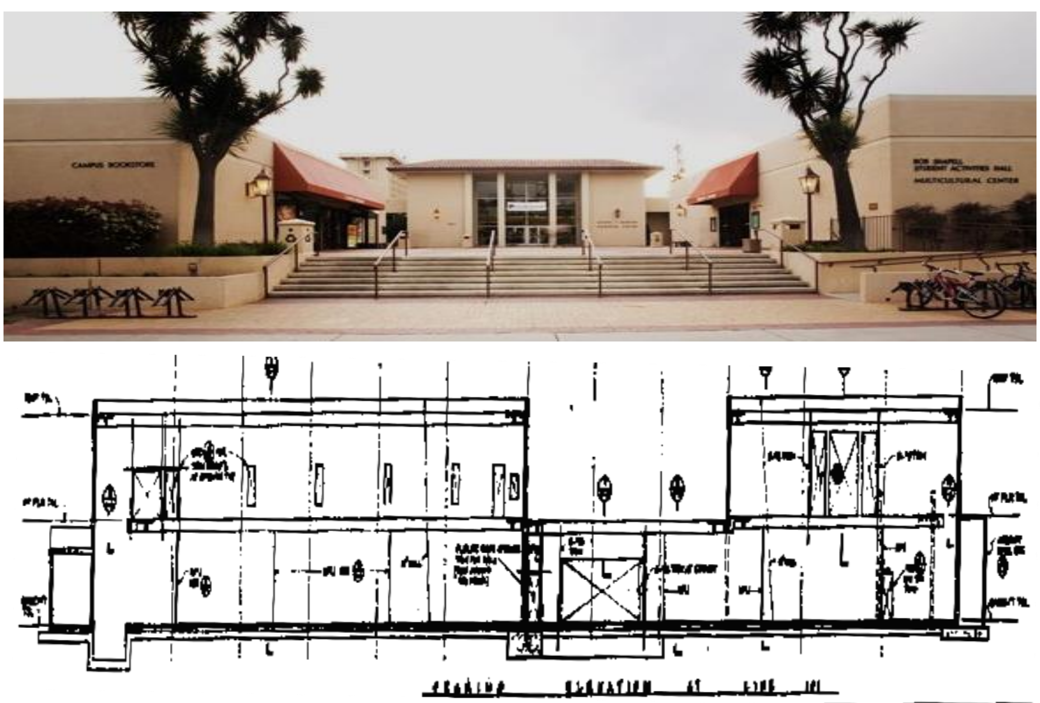


Figure 17. A comparison of the Benson Plaza with its framing elevation view.

As shown in the framing elevation view above, the MCC is located in the top right corner of the plaza, with the SCU Drahmann Tutoring Center located directly underneath. The basement, however, stretches across the entire plaza, and thus, the foundation supports needed for the renovation of the MCC must not interfere with the pre-existing structure.

The current dimensions of the top floor of the building, taken from the Santa Clara University's filed Benson Center architectural drawings, are as follows: 69 ft long x 65 ft wide x 15 high, with a total area of 4420 square ft. The perimeter of the building is comprised of 1-ft-thick structural concrete walls that, in addition to the columns, support the concrete roof.

3.2 Proposed Structural Design Plan

Given the constraints of the original space, a preliminary structural design plan was created to address the needs of the MCC while not disturbing the bottom story. Thus a new roof design will be put in place, inspired by Santa Clara University's Leavey Center.

The Leavey Center, formerly known as the Harold J. Taso Pavilion, was originally built in 1975 and boasted an air-supported fabric roof. This roof was in place for 25 years and was then deflated in 2000. The renovation of the Pavilion into the Leavey Center, included a new truss-joist-supported roof which was then supported by four large trusses held up by 8 columns placed along the exterior of the building.

The redesign of the MCC will closely follow the structural design of the Leavey Center. The first step of the MCC's structural redesign plan entails demolishing the cast-in-place concrete roof and top floor interior columns. The roof will then be replaced by a metal deck supported by prefabricated steel truss joists. Two large steel trusses spanning the entire length of the building will be placed along the top of the MCC to support the steel joists. Four concrete columns will be erected along the exterior of the building to support the steel trusses.

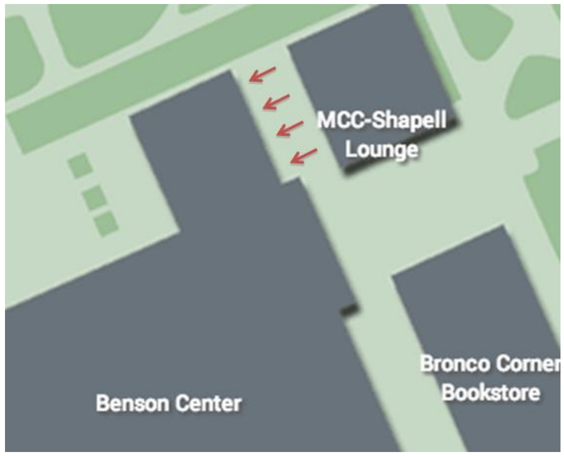


Figure 18. An illustration indicating that the MCC will be extended into the patio area in between the current MCC and the Robert F. Benson Memorial Center.

As thoroughly discussed in Chapter 2, it should be noted that the dimensions of the MCC will be expanded, as this will prove critical in the structural design of the renovation. The dimensions of the building prior to and proceeding the renovation are illustrated in the table below:

Table 1. A summary of the dimensions of the MCC before and after renovation.

	Current Space	Proposed Space
Length (ft)	65	80
Width (ft)	69	69
Height (ft)	15	15
Area (ft^2)	4485	5520

Using BIM Revit 2014, a 3D model incorporating all the elements of the proposed renovation of the MCC was created. Figure 19 below illustrates the proposed redesign of the MCC.

Figure 19. A 3D representation of the proposed renovation of the MCC created using BIM Revit 2014.

It should be noted that the proposed structural design plan below is a preliminary design and only includes a rudimentary overview of the necessary calculations and elements of the structure. Thus, this project only focuses on the selection of rough member sizes and does not delve into connection detailing. In addition, the existing building is assumed to be structurally sound without the heavy concrete roof in place. Thus, a seismic design plan pertaining to this renovation has also been omitted in this proposal.

3.3 Demolition

The first element of the MCC's preliminary structural redesign entails demolishing the cast-in-place concrete roof and the four interior concrete columns on the top floor. The original concrete roof consisted of a concrete waffle slab, as illustrated in the figure below taken from the original drawings for the Benson Center as provided by the SCU's Department of Planning and Projects.

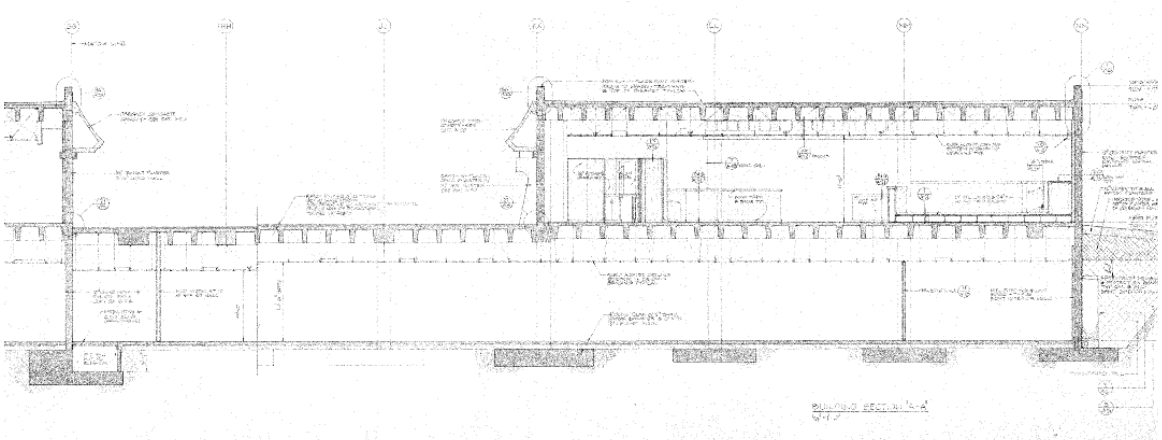


Figure 20. An elevation view of the Benson Plaza highlighting the waffle slab concrete roof of the MCC.

This waffle slab roof design adds a vast weight onto the structure, and thus needs to be supported by four interior concrete columns in addition to the structural concrete walls. However, because the main objective of the renovation aims to provide a more functional and open layout for the members of the MCC, the four interior columns on the top floor must be removed. With the removal of the columns, the heavy concrete weight will also need to be modified into a lighter roof design plan, which is described in the process below.

3.4 Metal Deck

The first element of the MCC's preliminary structural redesign following the demolition of the top-floor concrete columns and cast-in-place concrete roof involves designing the metal deck highlighted in blue in Figure 21 below.

Figure 21. A 3D capture of the renovated MCC, highlighting the metal deck roof.

3.4.1 Distributed Loads

The first step in the metal deck design was establishing expected distributed loads applied to the roof. Based on the standard values outlined by the California Building Code (CBC) Sec. 1607.1, the design live load was determined to be 20 psf. The expected total dead load was roughly estimated to be around 24.4 psf. A breakdown summary of the distributed loads is presented in the table below.

Table 2. A summary of the expected distributed loads on the MCC roof.

Distributed Load	**Load (psf)**
LIVE, *LL*	20
DEAD, *DL*	
- *Mechanical/Electrical*	8
- *Fireproofing*	2
- *5-ply Gravel*	6.5
- *Suspended Ceiling*	2
- *Insulation*	2
DEAD total	20.5

A metal deck that would span the area of the roof was then selected based on its ability to support the sum of these initial loads. The total factored load was then calculated using the equation

$$P_u = 1.2DL + 1.6LL \qquad \text{(CBC 1606.1)}$$

where DL stands for total dead load and LL denotes total live loads. Thus, given Equation CBC 1606.1, the factored load applied to the metal deck was calculated to be 56.6 psf. This value was then compared to the allowable un-factored loads applicable to metal deck products supplied by Verco Decking, Inc.

3.4.2 Metal Deck Selection

A product catalogue provided by Verco Decking, Inc. was used as a reference for the metal deck selection, resulting in the selected use of the PLN3 deck, illustrated below.

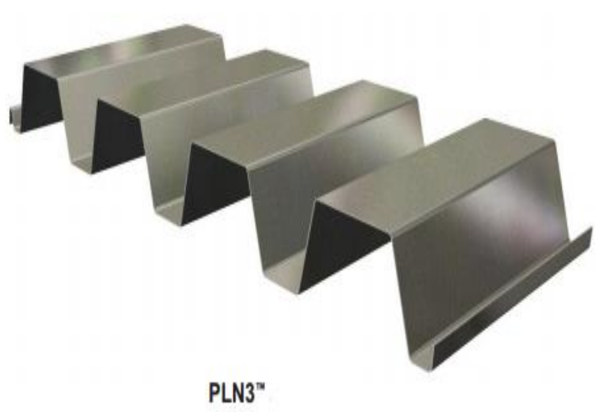


Figure 22. The metal roof deck PLN3 provided by Verco Decking, Inc. to

Material properties of the deck are listed in the Appendix, but a brief summary is presented below:

Table 3. A summary of the selected PLN3 metal deck provided by Verco Decking, Inc.

Metal Deck PLN3	
Deck Gage	20
Steel Type	ASTM A992
Span (ft)	10
Allowable Load(psf)	90
Weight (psf)	2.9

Thus, from the properties provided by the Verco Decking, Inc. product catalog, it was established that the Metal Deck PLN3 is suitable to uphold the applied uniform load of the roof of 56.6 psf, as the PLN3 has a greater allowable load of 90 psf.

Given this selection, the dead loads and uniform load of the roof were then updated to include the weight provided by the PLN3 metal deck. Table 4 below indicates the final distributed loads expected on the roof that will then applied to the prefabricated steel truss joists used to support the metal deck.

Table 4. A summary of the distributed loads to be applied on to the steel truss joists used to support the metal deck.

Distributed Load	**Load (psf)**
LIVE, *LL*	20
DEAD, *DL*	
- *Mechanical/Electrical*	8
- *Fireproofing*	2
- *5-ply Gravel*	6.5
- *Suspended Ceiling*	2
- *Insulation*	2
- *Metal Deck*	2.9
DEAD total	22.9
Factored Load, P_u (P_u = 1.2DL + 1.6LL)	59.5

3.4.3 Steel Truss Joists

Truss joists were then selected as the next components of the roof as illustrated in blue in Figure 23 below.

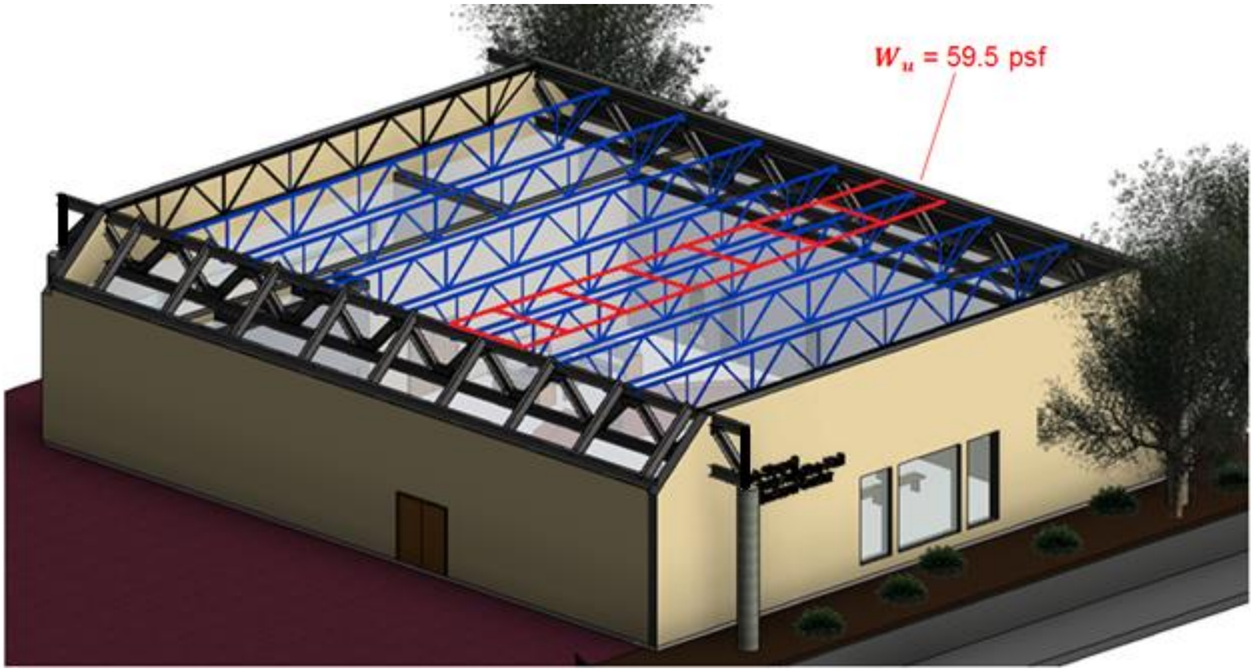


Figure 23. A 3D capture of the renovated MCC, highlighting the steel truss joists.

As mentioned previously, the expected roof loads will apply a factored load of 59.5 psf that will then be translated onto the steel truss joists. It is critical that the selected prefabricated steel truss joist can sustain this applied load. However, before a selection can take place, a summary of the truss joist dimensions, placements, and characteristics must first be established.

As illustrated in the final design, nine steel truss joists will be used to support the metal deck. Each 60-ft truss joist will be spaced 10 feet apart from one another and will thus be evenly distributed across the 80-ft length of the MCC, which includes the extension previously mentioned. Given this even distribution, the width of load affecting each truss joist, also referred to as the tributary width, t_w, was then determined for both interior truss joists and exterior truss joists. Because the interior truss joists were exactly 10 feet apart from one another, each interior joist had a t_w of 10 ft, whereas each exterior truss joist only needed to support half the width of the interior, thus resulting in a t_w of 5 ft.

These tributary widths were then multiplied by the applied factored roof load, P_u, of 59.5 psf in order to determine the load applied along the length of each joist. Table 5 below summarizes the load per linear foot onto both the interior and exterior truss joist.

Table 5. A summary of linear loads applied on to the steel truss joists.

Truss Joist Type	**Uniform Linear Load, Wu** **Wu = tw x Pu**
Interior	595 lb/ft
Exterior	297.5 lb/ft

The figure below indicates how the roof uniform load, pw_u, will be distributed on to one interior truss joist.

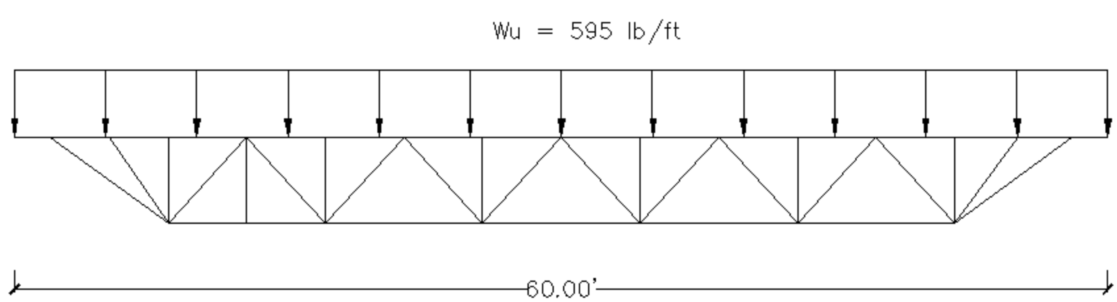


Figure 24. An interior steel truss joist with an applied uniform load of 595 lb/ft.

Once the uniform allowable loads applied onto each joist were established, a prefabricated steel truss joist type was then selected from the New Millennium Building Systems Product Catalog. *[Note: also check if the joist catalog uses factored or "allowable" loads]* Given a maximum applied linear load of 595 plf, it was determined that a deep long-span truss joist type 56DLH11 was the most ideal joist, as it could withstand a maximum applied load of 613 plf, which is greater than Wu. In order to remain consistent, the 56DLH11 will be used for both the interior and exterior truss joist.

Based on the product catalog, the 56DLH11 has an approximate weight of 26 plf. Thus, when multiplied by its length of 60 ft, each individual truss joist will have a self-weight of 1560 lb. Given its self-weight and Wu, the reaction was then determined to be 18.63 kips for each interior truss joist and 9.32 kips for each exterior truss joist. A more detailed calculation regarding the truss joist shear force can be found in the Appendix. These reactions will then be applied onto the steel trusses.

3.4.4 Angled Roof Ends

The two gaps between the large trusses and the existing concrete walls on the north and south sides of the building will be framed with small beams that will support an angled steel deck roof, highlighted in blue in Figure 25 below.

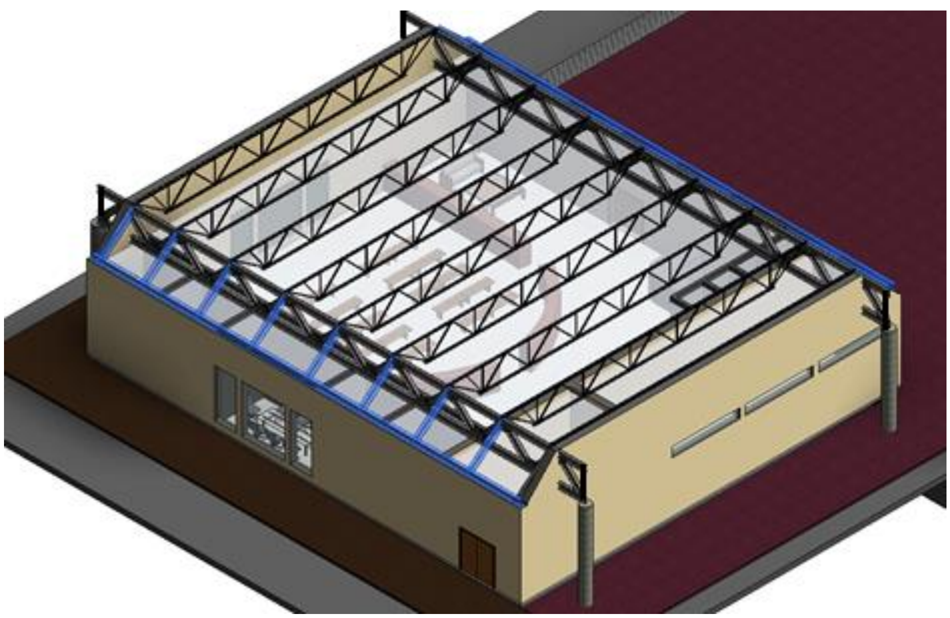


Figure 25. A 3D model of the MCC, highlighting the beams and girder used to support the angled roof ends.

3.4.5 Angled Beam and Girder Selection

In order to select the proper beam needed to support the angled roof ends of the structure, the uniform roof loads were first determined. Identical to the uniform loads applied onto the steel truss joists, each angled beam will experience a uniform load of 595 lb/ft. Calculations were then done to calculate the moment demand, deflection limits, moment capacity and shear capacity for each beam. Then using the American Institute Steel Construction Code (AISC), it was determined that a W8x10 would be most ideal to support the loads of the roof. The girder selected will be placed on top of the existing concrete wall as support. This is picture below in Figure 26.

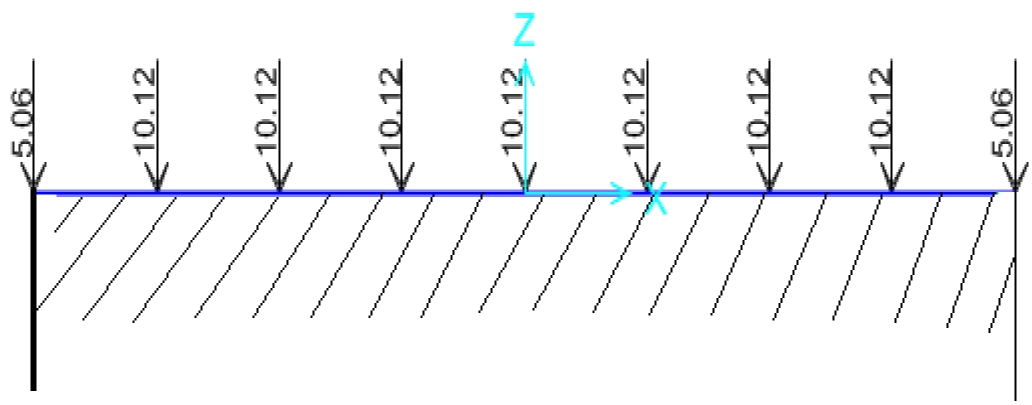


Figure 26. A free body diagram of the girder supporting the angled roof ends

The girder is a necessary component for this project with respects to the extension. We have recognized that a girder would not be needed for the angled beams to rest on, and that the original concrete walls would provide spots for load distribution. However, the extension does not have a concrete wall for the angled beams to rest on, so a cantilever girder will be placed for the angled beams on the extension part of the project.

3.4.6 Steel Trusses

In order to support the reactions from the steel truss joists, two steel trusses were then designed to be placed directly underneath the joists and angled beams. However, unlike the Leavey Center whose steel trusses are placed along the outside perimeter of the building, the two steel trusses in the MCC renovation will be placed on top of the structure offset by five feet towards the interior of the building as illustrated in Figure 27 below.

Figure 27. A 3D capture of the renovated MCC, highlighting the steel trusses.

This was done in order to allow the columns foots supporting the columns that ultimately hold up each steel truss to be placed in soil that would not directly impact the basement underground. More information regarding this can be found under the Column Footings section.

The finalized steel truss design consisted of two steel trusses each with a depth of 5 ft and a length of 90 ft. Each truss will then experience 9 point loads applied by the ends of each truss joist. The reaction from each interior truss joist will be 9.32 kips and each exterior reaction will be 4.66 kips. The reaction from each angled beam will be 2.53 kips for each interior beam and 1.26 kips for each exterior beam. Combining the two loads gives a total interior point load of 11.85 kips and a total exterior point load of 5.92 kips, as illustrated in Figure 28 below.

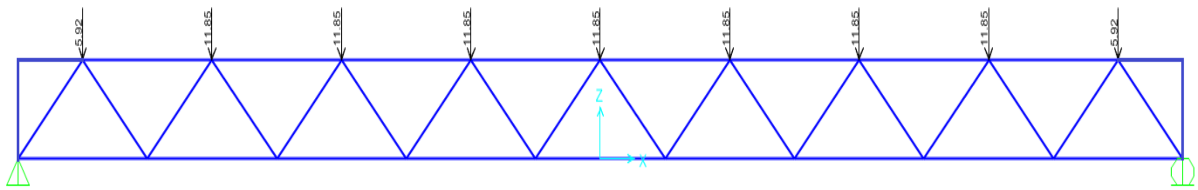


Figure 28. Total point loads applied onto each steel truss.

Once these point loads were established, computer program SAP 2000 was used to analyze the truss under loading. From there, ENERCALC was used to verify maximum axial and bending stress ratios as well as allowable moment based on various member sizes. By using LFRD design, many iterations were done between these two programs to find ideal steel truss member sizes Two steel member sizes were selected to provide for the most cost effective solution and lightest truss design with respects to the load demands on the truss. The top and bottom of each truss will be made of hollow structural steel member HSS9x9x$\frac{1}{2}$ and each angled interior member of the truss will be a double-angled LL$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{5}{16}$x$\frac{3}{8}$ member, as shown in the image below.

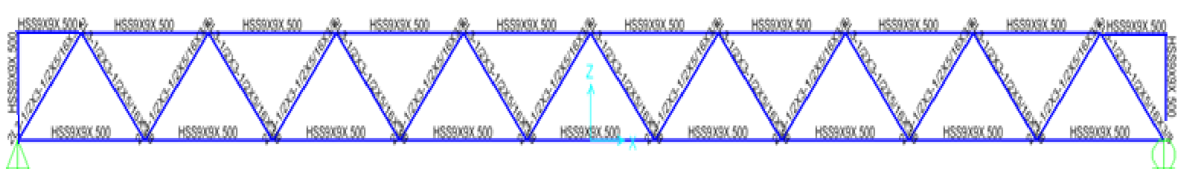


Figure 29. A SAP 2000 illustration of the steel truss and its selected steel members.

The self-weight of each steel truss member as well as the applied point loads yields a reaction force at each end of the truss of 36.25 kips, as illustrated below. Both factored loads and self-weight were used in our calculations.

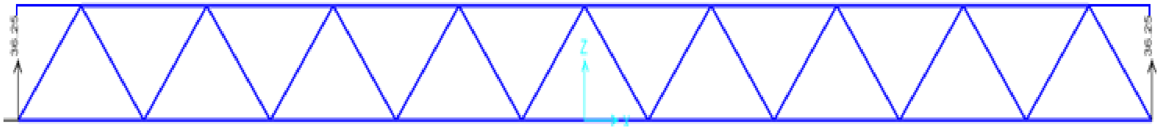


Figure 30. A SAP 2000 illustration of the steel truss and its reaction forces.

This reaction force will then be applied to the concrete columns placed along the exterior of the structure.

3.4.7 Exterior Columns

In order to hold up the steel trusses, four concrete columns will be erected along the perimeter of the structure directly underneath each steel truss end shown in Figure 31 below.

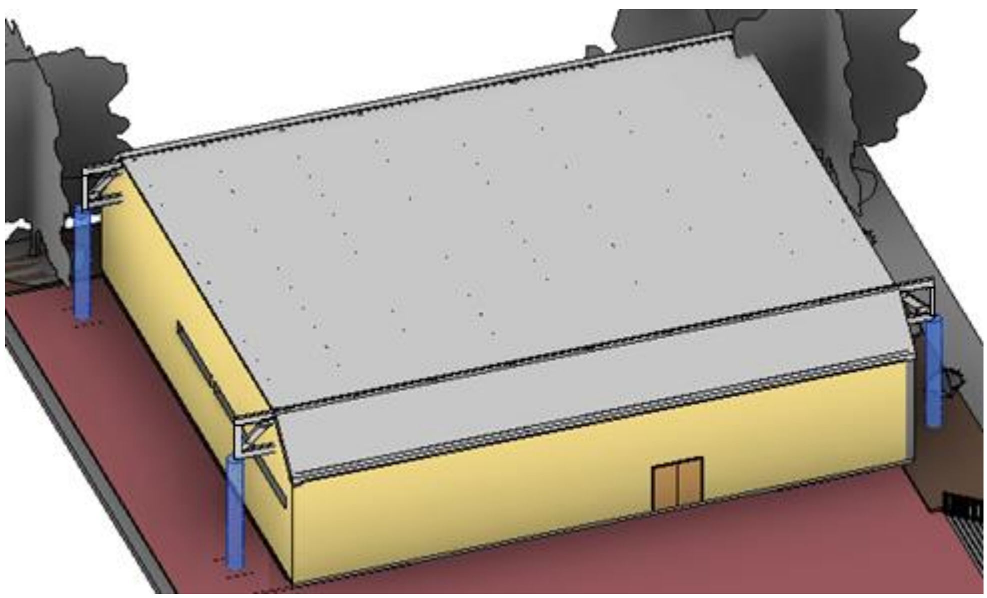


Figure 31. A 3D capture of the renovated MCC, highlighting the exterior concrete columns.

As mentioned previously, each concrete column will be offset five feet from the length and width of the structure towards the interior in order to accommodate the basement underneath. Design of the concrete columns was not included in the project scope, since it would be governed by seismic design (not part of this project). A rough estimate of 24" diameter column was used for the foundation design and cost estimate. Therefore, with a height of 15 ft, a diameter of 2 ft, and a concrete density of 150 pcf, each column will yield of weight of 7070 lb that will then be applied onto a column footing.

3.4.8 Column Footings

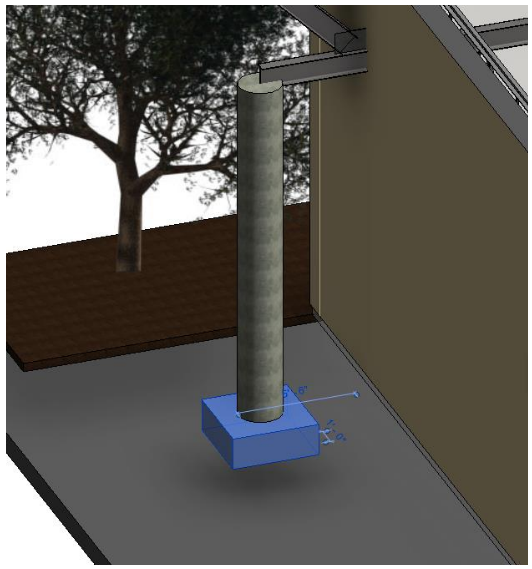


Figure 32. The underground column footing used to support the concrete column.

Directly underneath the MCC is the SCU Drahmann Tutoring Center, with surrounding hallways connecting this space to the basement of the Robert F. Benson Memorial Center. Thus, the footings needed to support the concrete columns needed to be placed in an area that would not disturb the Benson Basement and will consequently be placed 5 feet from the length of the building towards the interior.

For the preliminary footing size, the applied dead and live loads were used. The reaction forces of the truss provided a load of 36.25 kips. The self-weight of the column provided a force of approximately 7.07 kips onto the footing. These two loads provided a total load of 43.32 kips onto each footing.

The geotechnical report of the Benson Plaza was used as a reference to determine the allowable bearing pressure of the soil in which the footing will be placed. According to the report, the allowable soil bearing pressure due to both dead and live loads was stated to be 2500 psf. This pressure and along with an applied total load of 43.32 kips will thus call for a minimum footing size of 18.1 ft^2, and rounded to the nearest half-foot yields a footing size of 4.5 ft x 4.5 ft. For the cost estimate, an estimated footing depth of 3 ft was used. Figure 33 below illustrates the final column footing size below.

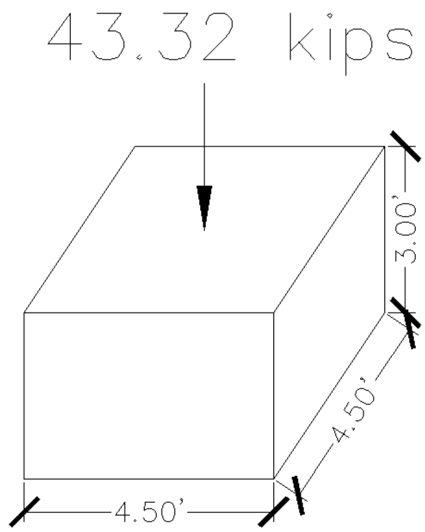


Figure 33. Concrete column footing size.

4 Construction Management

4.1 Work Breakdown Structures (WBS)

A Work Breakdown Structures (WBS) was generated to highlight project deliverables needed in order to fulfill the requirements of the renovation. The MCC redesign contains three main subcategories for construction. These elements include the demolition processes, the structural implementation, and the architectural design. All three components are dependent on each other, with respect to the order of certain construction elements. Below is an example showing the 3 major components of the redesign, as well as an example of how the structural components are broken down in a sequential order.

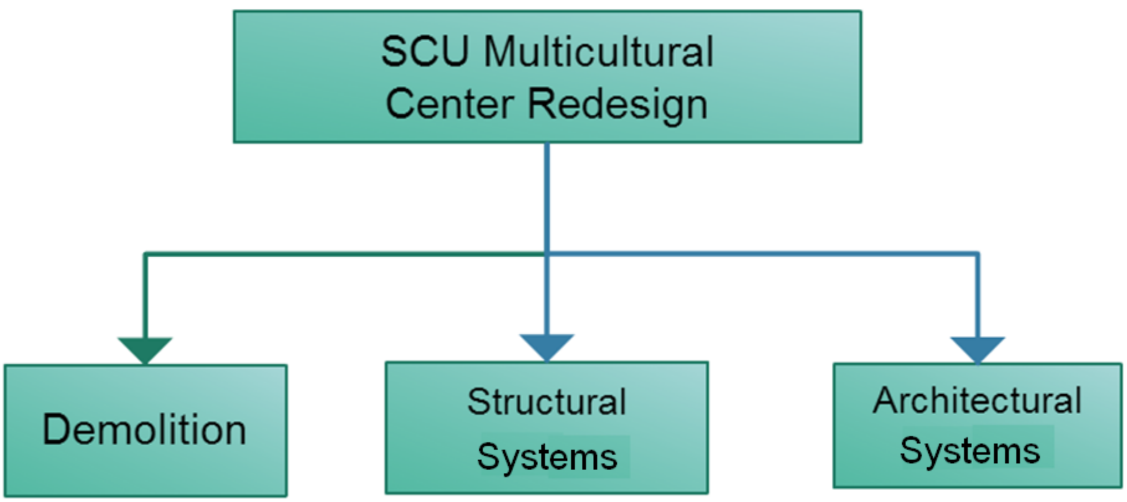


Figure 34. An example of a work breakdown structures that includes the main elements of the proposal pertaining to the entire redesign.

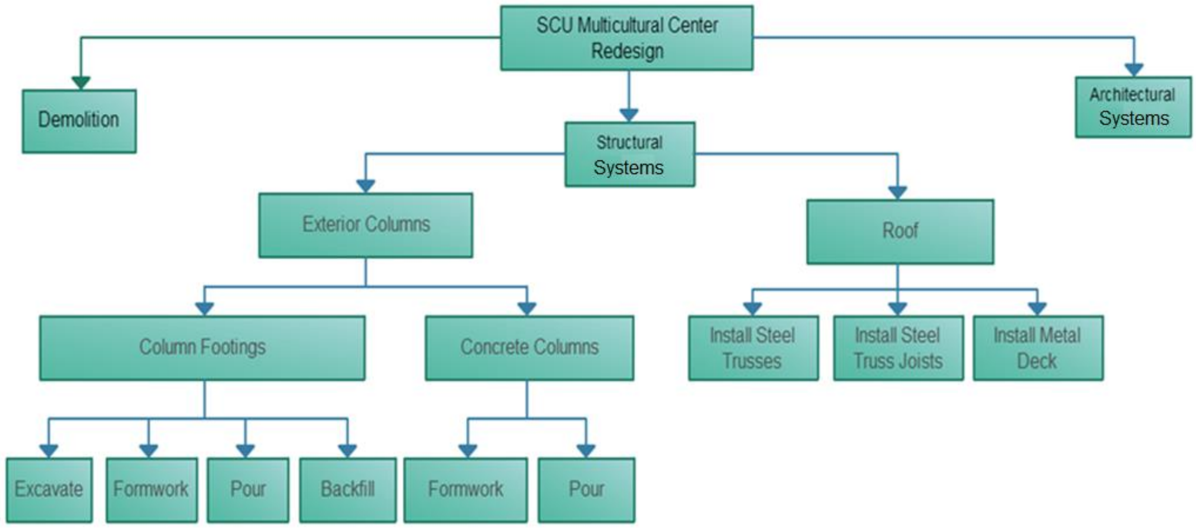


Figure 35. An image showing the structural elements organized in a work breakdown structure.

The image above takes a closer look at how the structural elements of the project were implemented in a sequential order. The two primary categories of the structural elements were the placement of the exterior columns and the roof systems. In order to implement the column footings, for an example, construction calls for excavation, formwork placement, pouring of concrete and backfill. These four items consist of developing the column footings. On top of the column footings there will be the actual concrete columns, which consist of formwork placement and concrete pouring. These elements combined together satisfy the exterior column prerequisites under the structural work breakdown tab. The other category, as mentioned, is the roof and its specific components. The roof system will be supported by both the steel trusses and the steel truss joists. The actual rooftop itself is the installation of the metal deck, which will go on last after the exterior columns and the trusses have been placed. This is just an example of one of the work breakdown structure tabs out of the three. The demolition and architecture work breakdown structures are pictured in the Appendix I.

4.2 Cost and Duration Estimation

The purpose of creating a WBS was to determine which action items were needed to complete the renovation. From there, given the action items established, a cost and duration estimate was then created in order to determine expected costs for the renovation and duration of individual activities.

Once each action item in the WBS was established, an activity list was then inputted into a Cost Estimate Excel spreadsheet template. The template assisted in breaking down the cost of each activity given the renovation's required quantity. Average values taken from *the RSMeans Building Construction Cost Data 2014* and *RSMeans Square Foots Costs 2015* were inputted into the template to estimated costs for each activity. The Cost Estimate template was broken down into two main sections. Section 1 dealt with highlighting the material costs, labor costs, and equipment costs for each activity, whereas Section 2 incorporated the proper adjustment factors such as waste, tax rates, and city indexes for each line item. Snippets from Sections 1 and 2 of the Cost Estimate breakdown are presented below.

ID	Activity	Quantity	unit	Daily Output	Duration days	Material $/Unit	**WTC**	Material Cost	Crew	Labor $/Unit	**Labor FC**	Labor Cost	Equip. $/Unit	**Equip. C**	Equip. Cost	Total Cost	Means Item	Page
1	Metal Decking (Roof)	5520	S.F.	4300	1.28	1.47	1.041	$ 8,445	E-4	0.37	1.400	$ 2,860	0.03	1.077	$ 178	$ 11,483	05 31 23.50 2700	139
2	Steel Truss Joists (installation and material)	621	L.F.	2000	0.31	41	1.093	$ 27,823	E-7	1.95	1.400	$ 1,695	0.87	1.077	$ 582	$ 30,100	05 21 13.50 3260	136
3	Demolish Non-Structural Walls (Drywall)	23	Ea.	24	0.96		1.477	$ -	1 Clab	11.7	1.303	$ 351		1.002	$ -	$ 351	02 41 19.16 6100	32
4	Steel Trusses Material	11.26	ton			2137	0.957	$ 23,028								$ 23,028		
5	Steel Strusses Installation				1.00			$ -	E-7		1.30000	$ 5,643			$ -	$ 5,643		
6	Demolish Structural Columns	240	C.F.	11300	0.02		1.273	$ -	B-3	0.16	1.21000	$ 46	0.2	1.21	$ 58	$ 105	02 41 16.13 0600	30
7	Demolish Roof	7475	C.F.	11300	0.66		0.000	$ -	B-3	0.16	1.21000	$ 1,447	0.2	1.21	$ 1,809	$ 3,256	02 41 16.13 0600	30
8	Mobilize - Crane, up to 75 ton	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 75	62	1.091	$ 68	$ 143	01 54 36.50 2000	21
9	Mobilize - Forklift	0.5	Ea.	7.2	0.07		0.000	$ -	1 Eqhv	53	1.41830	$ 38	62	1.091	$ 34	$ 71	02 54 36.50 2000	21
10	Mobilize - Hydraulic Pump	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 75	62	1.091	$ 68	$ 48	02 54 36.50 2000	21
11	Demobilize - Crane, up to 75 ton	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 75	62	1.091	$ 68	$ 143	01 54 36.50 2000	21
12	Demobilize - Forklift	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 38	62	1.091	$ 34	$ 71	02 54 36.50 2000	21
13	Demobilize - Hydraulic Pump	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 25	62	1.091	$ 68	$ 48	02 54 36.50 2000	21
14	Structural Wall Reshoring	4020	S.F.	1400	2.87	0.5	1.266	$ 2,544	2 Carp	0.4	1.71210	$ 2,753		1.317	$ -	$ 5,297	03 15 05.70 1500	62
15	Column Foundation Excavation	13	B.C.Y	150	0.09				B-11C	4.36	1.30260	$ 74	2.23	1.317	$ 34	$ 108	31 23 16.13 0050	577
16	Column Formwork	480	S.F.C.A	216	2.22	1.41	1.268	$ 858	C-1	7.05	1.92270	$ 6,506		1.091	$ -	$ 7,365	03 11 13.25 6500	54
17	Concrete Columns	7	C.Y.	51.85	0.14	257	1.354	$ 2,437	C-14A	171	1.73030	$ 2,071	14.35	1.331	$ 134	$ 4,642	03 30 53.40 1400	72

Figure 36. A snippet of Section 1 of the Cost Estimate breakdown.

ADJUSTMENT FACTORS	Waste	Tax	Mat. City Index	Material WTC	Labor Overhead	Inst. City Index	Labor FC	Equip. C
Concrete Column	1.06	1.0875	1.175	1.354	1.3	1.331	1.7303	1.331
Metal Decking (Roof)	1	1.0875	0.957	1.041	1.300	1.077	1.400	1.077
Steel Truss Joists	1.05	1.0875	0.957	1.093	1.300	1.077	1.400	1.077
Demolish Non-Structural Walls	1	1.0875	1.358	1.477	1.300	1.002	1.303	1.002
Steel Trusses Material	1	1.0875	0.957	0.957	1.300	1.077	1.400	1.077
Demolish Structural Colums	1	1.0875	1.171	1.273	1.3	1.21	1.21	1.21
Demolish Roof	1	1.0875	1.171	1.273	1.3	1.21	1.21	1.21

Figure 37. A snippet of Section 2 of the Cost Estimate breakdown.

4.2.1 Line Items Example

In order to demonstrate the effectiveness of the Cost Estimate template, the process done in order to estimate the cost of Line Item 17, which is the implementation of the four exterior concrete columns, is summarized below:

Cost Estimating Spreadsheet (RSMeans Building Construction Cost Data 2012)																		
ID	Activity	Quantity	Unit	Daily	Duration Days	Material $/Unit	WTC	Material Cost	Crew	Labor $/Unit	Labor FC	Labor Cost	Equip. $/Unit	Equip. C	Equip. Cost	Total Cost	Means Item	Page
17	Concrete Columns	7	C.Y.	51.85	0.14	257	$1.35	2436.71	C-14A	171	$1.73	2071.169	$14.35	1.331	133.699	$4,641.58	03 30 53.40 1400	72

Figure 38. Cost estimate breakdown of the concrete column installation.

The RSMeans Building Construction Cost Data 2014 provided an estimate for the concrete columns formation and installation per cubic yard of concrete. Given a column height of 15 ft and diameter of 2 ft, an approximate total of 7 cubic yards of concrete will be needed to erect four concrete columns. The RSMeans then provided an average daily output of 51.85 cubic yards of concrete columns that are expected to be constructed per work day. Thus, a duration length for each activity can be determined by dividing the quantity by the daily output, which will prove useful later on upon the creation of a construction schedule for the project. The material cost, labor cost, and equipment cost for this line item were values taken directly from the RSMeans. However, adjustment factors were also taken into account in order to include waste, tax, labor overhead, material city indexes and installation city indexes, as shown in Section 2 of the Cost Estimate below:

ADJUSTMENT FACTORS	Waste	Tax	Mat. City Index	Material WTC	Labor Overhead	Inst. City Index	Labor FC	Equip. C
Concrete Column	1.06	1.0875	1.175	1.354	1.3	1.331	1.7303	1.331

Figure 39. Adjustment factors used to estimate the price of the concrete column installation

With an estimated 6% concrete batch waste, a San Jose tax of 8.75%, an estimated labor overhead of 30%, and material and installation city indexes provided by the RSMeans, adjustment factors were found for material, labor, and equipment that were then linked to Section 1 of the Cost Estimate breakdown, thus leading to total cost of $4,642 for the concrete column installation.

4.2.2 Final Cost Estimate

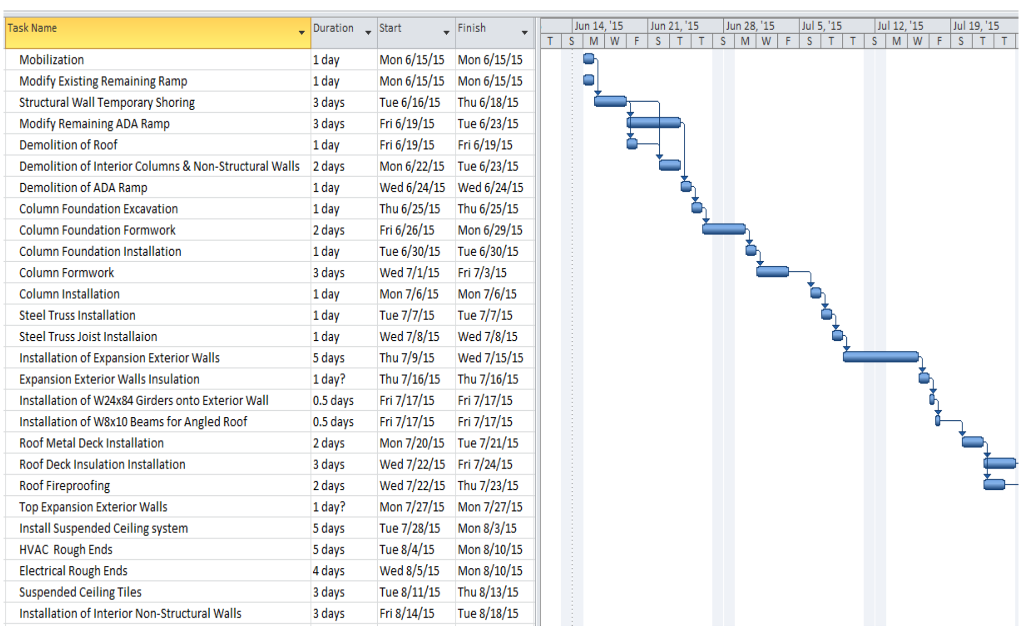
Once a complete list of nearly 60 line items were tabulated, which can be found in Appendix J, a total cost estimate was generated. The table below shows a breakdown of cost into the three main categories highlighted in the WBS: demolition systems, structural systems, and architectural systems. With the inclusion of overhead and profit, the total price of the renovation is expected to cost $864,341.00, and given a square footage of 5520 ft^2, the renovation will be expected to cost $156.58/square foot. A table summarizing the cost estimate breakdown is presented below.

Table 6. A preliminary cost breakdown for the proposed MCC renovation.

Demolition	$22,938.00
Structural	$115,884.00
Architectural	$547,163.00
Overhead & Profit	$178,356.00
Price	$864,341.00
Cost/Square Foot	$156.58

4.3 Scheduling

Along with the cost estimate and work break down schedules, the Multicultural Center Redesign includes an estimated construction schedule for the renovation. One of the more important aspects of the project redesign was to recognize that the construction would take place in the middle of campus, where student foot-traffic is heavy. With this in mind, we decided to implement a construction schedule designed specifically to minimize its impact with the student environment around the project location. The overall construction of the project is planned to take place during the summer quarter (June 15 – September 21) with a total construction time of eleven weeks. The construction would start the Monday, June 15, immediately following the 2015 undergraduate student commencement on Saturday. With the use of Microsoft project, we were able to organize a schedule that ensures each activity shall be completed within the summer months. The construction process for the entire building aims for a total of eleven weeks (ending August 28), which gives a 3 week contingency for any delays or unexpected challenges that require the construction schedule to extend. A small example of the scheduling is displayed in Figure 40 below, highlighting the expected start, end and duration times for each line item



Task Name	Duration	Start	Finish
Mobilization	1 day	Mon 6/15/15	Mon 6/15/15
Modify Existing Remaining Ramp	1 day	Mon 6/15/15	Mon 6/15/15
Structural Wall Temporary Shoring	3 days	Tue 6/16/15	Thu 6/18/15
Modify Remaining ADA Ramp	3 days	Fri 6/19/15	Tue 6/23/15
Demolition of Roof	1 day	Fri 6/19/15	Fri 6/19/15
Demolition of Interior Columns & Non-Structural Walls	2 days	Mon 6/22/15	Tue 6/23/15
Demolition of ADA Ramp	1 day	Wed 6/24/15	Wed 6/24/15
Column Foundation Excavation	1 day	Thu 6/25/15	Thu 6/25/15
Column Foundation Formwork	2 days	Fri 6/26/15	Mon 6/29/15
Column Foundation Installation	1 day	Tue 6/30/15	Tue 6/30/15
Column Formwork	3 days	Wed 7/1/15	Fri 7/3/15
Column Installation	1 day	Mon 7/6/15	Mon 7/6/15
Steel Truss Installation	1 day	Tue 7/7/15	Tue 7/7/15
Steel Truss Joist Installaion	1 day	Wed 7/8/15	Wed 7/8/15
Installation of Expansion Exterior Walls	5 days	Thu 7/9/15	Wed 7/15/15
Expansion Exterior Walls Insulation	1 day?	Thu 7/16/15	Thu 7/16/15
Installation of W24x84 Girders onto Exterior Wall	0.5 days	Fri 7/17/15	Fri 7/17/15
Installation of W8x10 Beams for Angled Roof	0.5 days	Fri 7/17/15	Fri 7/17/15
Roof Metal Deck Installation	2 days	Mon 7/20/15	Tue 7/21/15
Roof Deck Insulation Installation	3 days	Wed 7/22/15	Fri 7/24/15
Roof Fireproofing	2 days	Wed 7/22/15	Thu 7/23/15
Top Expansion Exterior Walls	1 day?	Mon 7/27/15	Mon 7/27/15
Install Suspended Ceiling system	5 days	Tue 7/28/15	Mon 8/3/15
HVAC Rough Ends	5 days	Tue 8/4/15	Mon 8/10/15
Electrical Rough Ends	4 days	Wed 8/5/15	Mon 8/10/15
Suspended Ceiling Tiles	3 days	Tue 8/11/15	Thu 8/13/15
Installation of Interior Non-Structural Walls	3 days	Fri 8/14/15	Tue 8/18/15

Figure 40. The proposed schedule of the project developed on Microsoft project.

4.4 Site Logistics

The site logistics that were addressed in the project included site access for the construction duration and proper temporary fencing for staging. The project site will be accessed from the corner of Alameda and Market Street along one of the pre-existing sidewalks located in front of the campus bookstore. Highlighted in yellow in Figure 41 below is the path to be taken in order to access the project site.

Figure 41. A plan view of how the project site will be access, via the intersection of Alameda and Market streets.

The second element of the site logistics plan includes implementing a proper fencing for the project proposal. The fencing will have a minimum of 12 feet clearance from the exterior of the building and will extend into a part of the Benson Mall lawn towards the north end of the building to allow space for project staging. Since the construction of the project will take place on a functioning center below (Drahmann Center), it was important to acknowledge and preserve the emergency exit path for the students that use the center below for emergency situations. The image on the left of Figure 42 shows Phase I of the fencing parameters. During Phase I, the extension of the building has not been constructed, which allows users of the Drahmann Center downstairs to access the stairwell to exit safely away from the building. The problem we recognized with the implementation of the extension is that the emergency exit will be covered and blocked off by the construction. The fencing will need to be adjusted to encompass the project with respects to the boundary of the building and the emergency exit to ensure the safety and availability of the inhabitants downstairs. For this, we suggested implementing a temporary path and access point as pictured in the right image of Figure 42. The fencing is mapped in two stages according to the progress of the project and its construction.

Figure 42. Mapping of phase one fencing (left) and phase two fencing (right) with repsect to emergency exit access for the downstairs Drahman Center and the implementation of the extension for the project proposal.

5 Ethical Concerns

5.1 Social Justice

Social Justice is our primary ethical issue of concern. The argument against the proposed project is that the University may not want to prioritize the issues according or relating to the MCC in lieu of other needs that the campus desires. The redesigning of the MCC will only be affecting a fraction of the student body that actually use this space for their own benefit. However, we argue that Santa Clara University, according to its ethical standards, must equally represent all academia, extracurriculars, and other forms of campus recreation alongside with representing all forms of culture and ethnical backgrounds to contribute to the University's standard of inclusive excellence. Thus, it is important to uphold social justice for all students on campus, regardless if they choose or not to choose to be utilizing certain spaces on campus, particularly the MCC.

Equal representation for all parties at SCU results in equal opportunity and representation of on-campus structures that house activities of particular usage according to the needs of the University and its student body. The MCC ensures an equitable distribution of benefits for those that are involved deeply in their own culture, and it is important to preserve the notion of this idea. Currently, the MCC needs improvement to better suit this important cultural necessity. The impact of this project on the overall character of the affected community will greatly improve representation of the student body who value their cultural standards and traditions, thus supporting social justice and improving Santa Clara University's idea of perspectives and inclusivity.

6 Relevant Non-Technical Issues

6.1 Political

Because our project consists of redesigning a building to be placed on Santa Clara University's campus, we must keep in mind certain local independent groups that endorse and support the construction of buildings on campus. Since the University is indeed private, most of the buildings on campus are built from donation-based efforts and funds from previous alumni and/or larger known alumnus families and organizations. Also, in order to build or reconstruct facilities on campus, the University board must approve all changes done to the campus regarding construction or maintenance on buildings. University approval is needed. Of course, there will be specific permitting requirements to be addressed through the University and the selected firm to construct the new building.

6.2 Environmental

With the construction of this environment, we do not see any problems with it affecting the surrounding environment or foundation. Since there is already a building located on the project site, there will be no need for further University excavation standards to be passed. General construction emissions will be taken into considerations and limitations will be provided, as with all construction-based projects. Regarding the social environment of the University, the project will benefit a specific (cultural) demographic at Santa Clara and will also benefit the student body as a whole, thus collectively improving the academic environment at the University.

6.3 Economic

We have realized the most cost-effective plan for the project is to leave the current building as it is and make smaller renovations inside the interior of the building. However, the chosen project seems applicable to the University's needs. The University also seems to be in good standing regarding its economic status in regards to on-campus construction, considering they have recently built a new residence hall in such a small time period to satisfy the needs of the growing population of student enrollment. This growing population of student enrollment can also entice the thought of the expansion of student resource centers, such as the MCC.

6.4 Safety

There will be construction related issues regarding our project in the time of the project and how it will affect the surrounding environment. Since the construction of this project will take place on campus, we have recognized that it would be best to keep construction timing in the summer, as opposed to the school year to decrease the chance of student foot traffic through the construction phases of the project. Also, general construction safety (OSHA requirements) will be implemented and the project must meet ADA requirements for access. Fencing and space limitations will be required around the project site proposal. Noise pollution will also be monitored around the site for students and the general public that access the university during the summer quarter.

6.5 Aesthetics

The project must correspond with the University's architectural standards and code. The building design must fit and be similar to the aesthetics of the surrounding buildings to keep the "mission style" theme of the University (ex: adobe wall colorings, tiled roofing, window outlet designs). The aesthetics of the building will not have an effect on the interest of the student users but rather if it can be passed by University standards. The aesthetics of the inside of the building, however, must appeal to student users to help promote the fluidity and adaptability of the building alongside its functionality. The internal design of the building is one of our main design focuses to create a more welcoming and useful environment for student users.

7 Conclusions

The Santa Clara University Multicultural Center Redesign project's main goal is to acknowledge the student growth on campus for future years to come and to design a new Multicultural Center that can accommodate the population on campus that utilize the Multicultural Center for its many uses. The implementation of this project contributes to the University's goal of inclusive excellence, as it is one of the University's standard to uphold multicultural and student diversity within the campus. The Santa Clara University Multicultural Center Redesign plans to renovate the current building both architecturally and structurally and to implement an extension to contribute to expansion for the facilities that are utilized there. The redesign includes a new metal deck roofing system supported by steel truss joists, large steel trusses, and exterior concrete columns. The building's pre-existing cast in place concrete shell will be relieved of its original roof load to help increase the building's stability against seismic activity. We hope that the redesign of the building not only accommodates the student growth on campus but also fosters an environment that is both safe, with respects to structural stability, and conducive to its purposes for the students that use the building with respect to club activities, general meetings and student study environments and learning.

Figure 43. Final exterior model of the redesign of the MCC.

8 References

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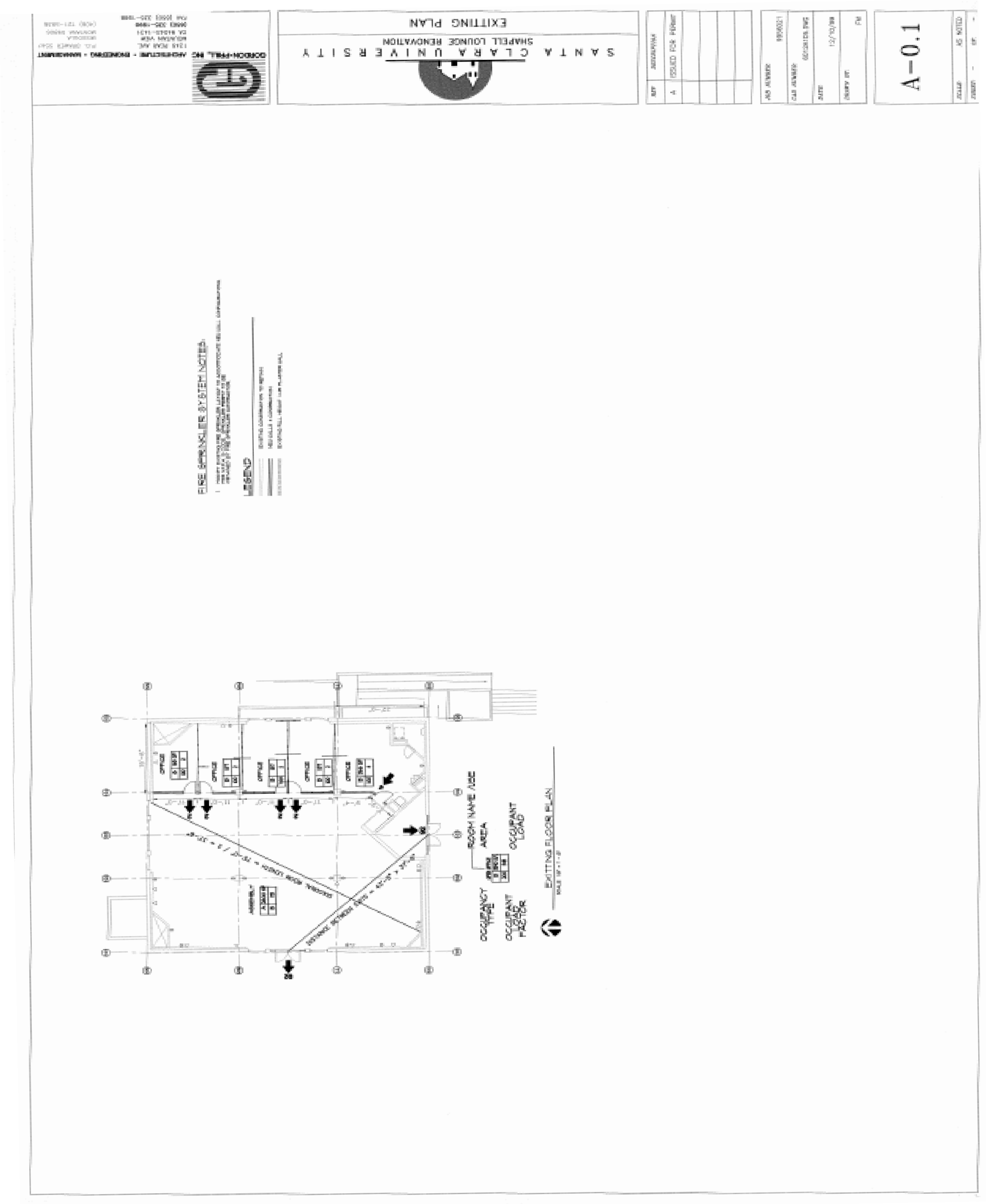
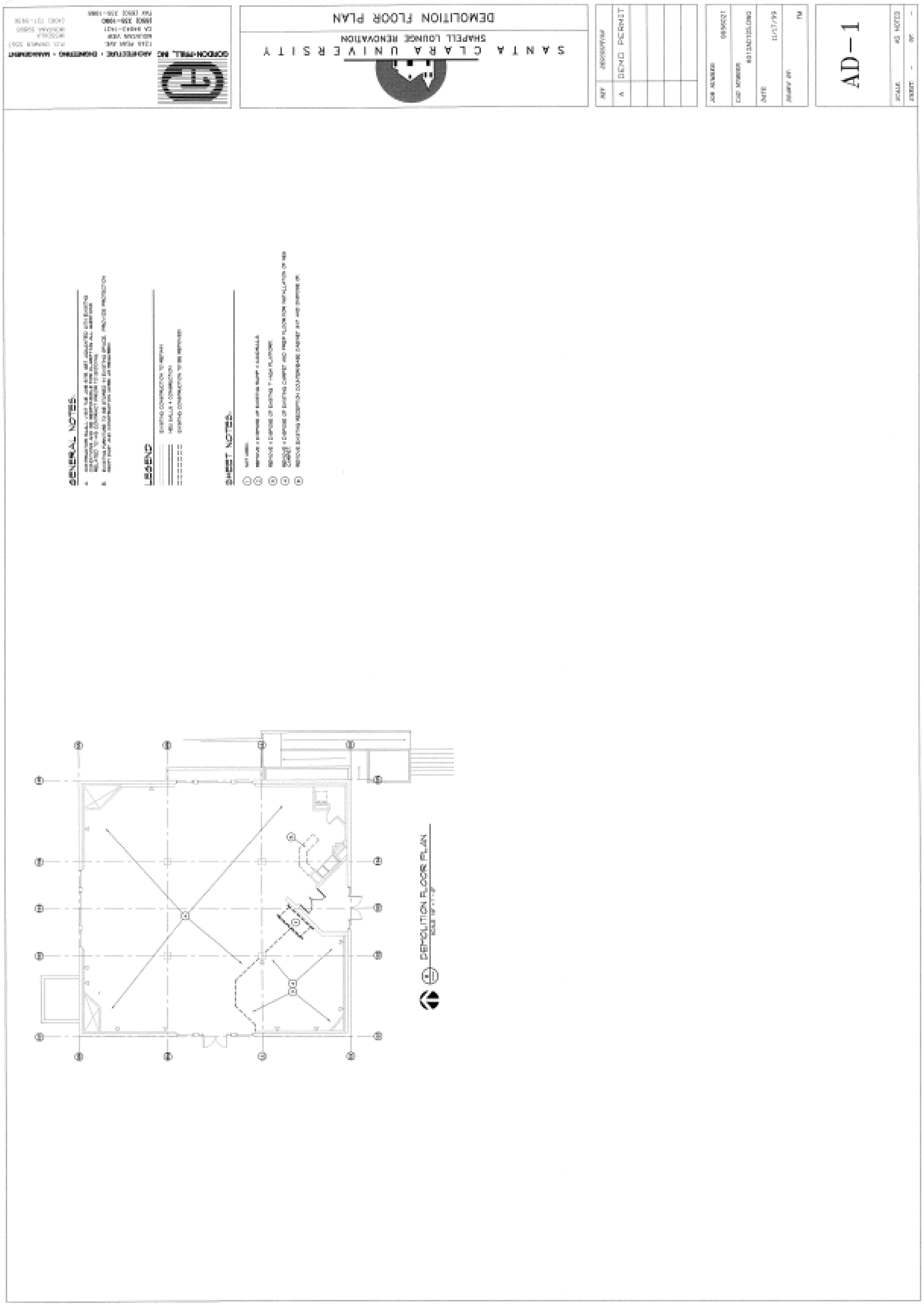
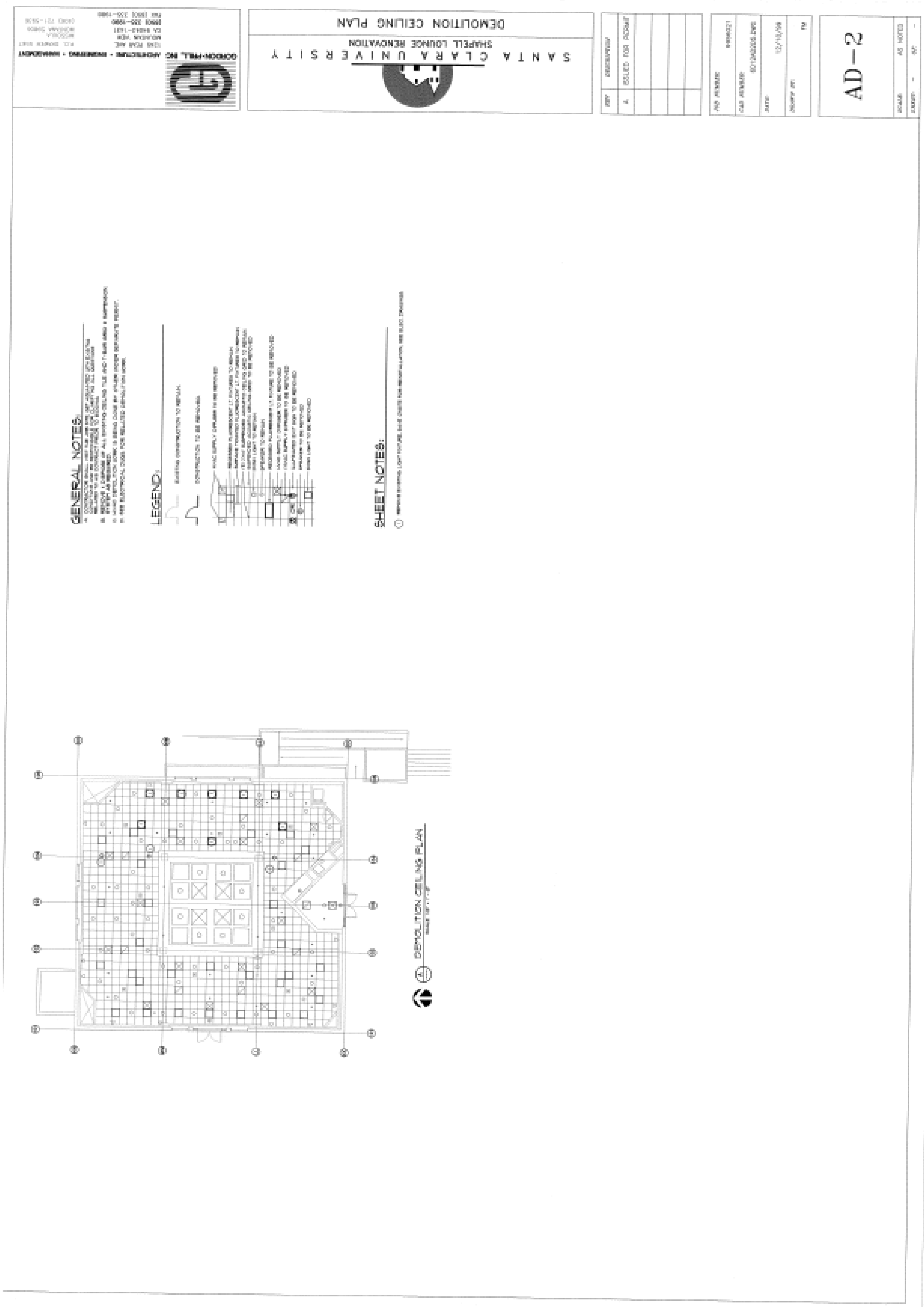
*Steel Construction Manua*l. 14th ed. Chicago: American Institute of Steel Construction (AISC), 2014. Print.

Appendix A

The architectural drawings of the original Bob Shapell Student Activities Hall, as provided by the Santa Clara Facilities Department




DEMOLITION FLOOR PLAN
SANTA CLARA UNIVERSITY
SHAPELL LOUNGE RENOVATION
DEMO PERMIT
AD-1
GENERAL NOTES
LEGEND
SHEET NOTES
DEMOLITION FLOOR PLAN



SANTA CLARA UNIVERSITY
SHAPELL LOUNGE RENOVATION
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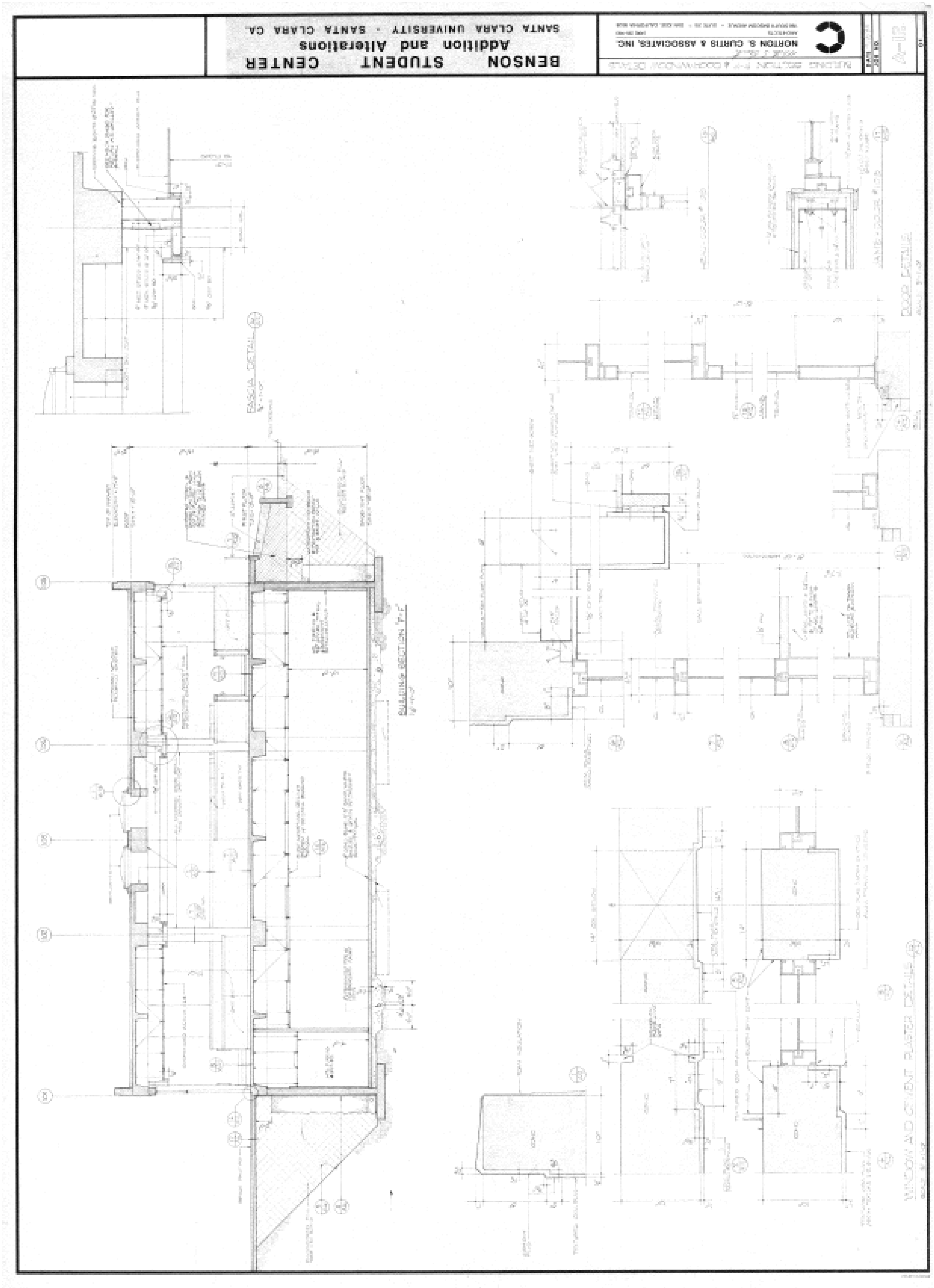
AD-2
GENERAL NOTES:
LEGEND:
SHEET NOTES:
ISSUED FOR PERMIT
12/10/98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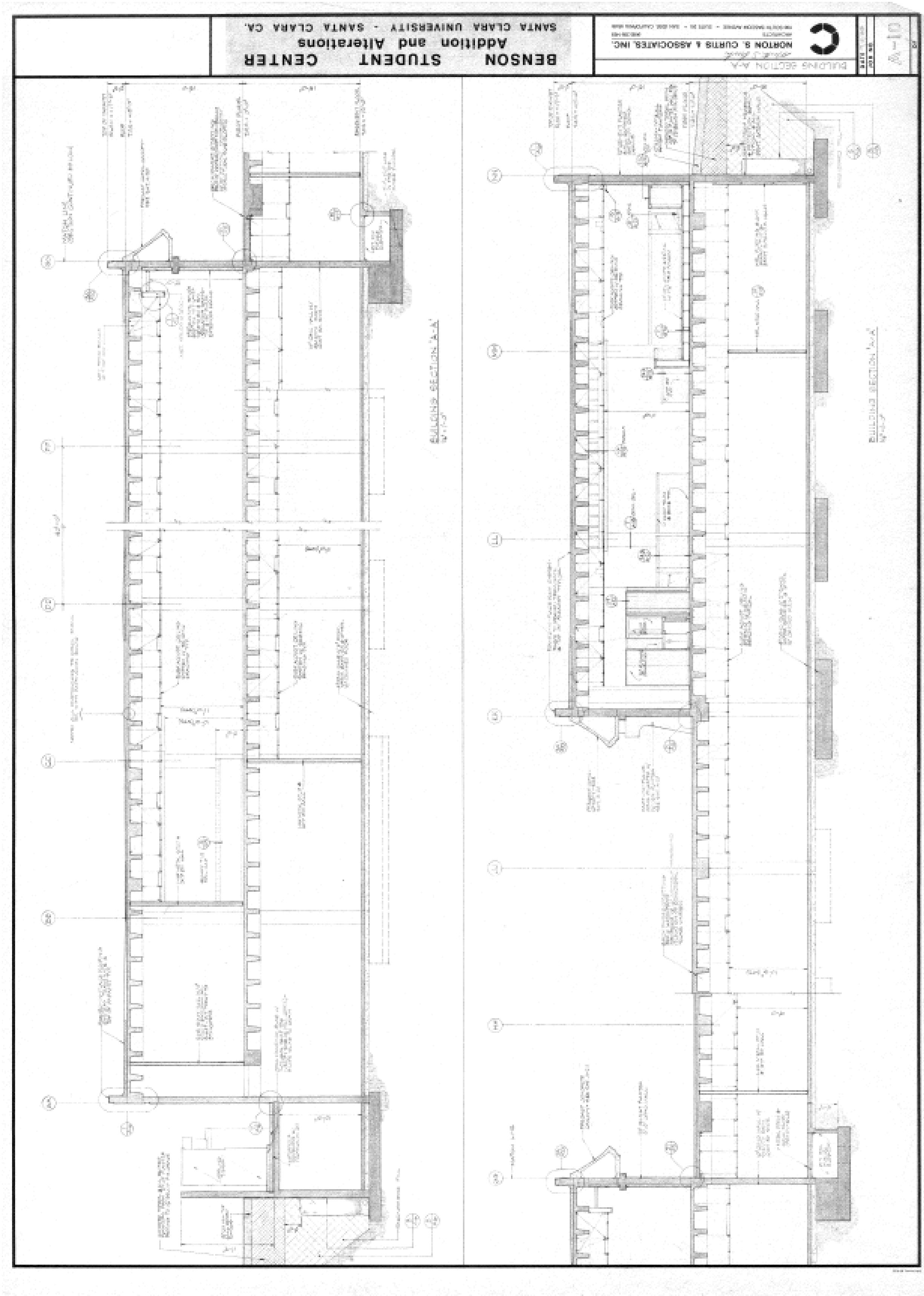
FASCIA DETAIL
BUILDING SECTION "F-F"
DOOR DETAILS
WINDOW AND CEMENT PLASTER DETAILS
A-12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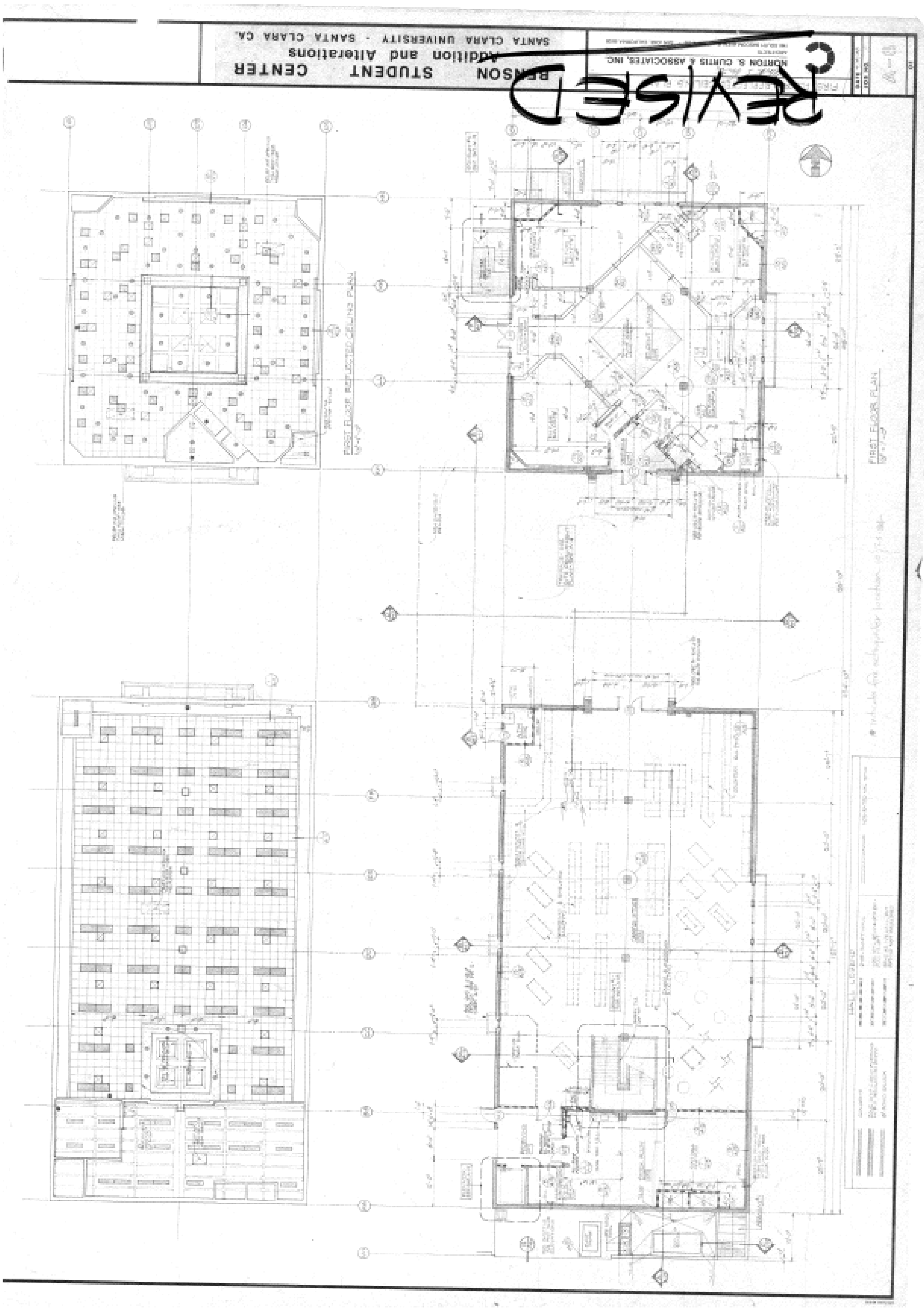
BUILDING SECTION A-A
A-10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SANTA CLARA UNIVERSITY - SANTA CLARA CA.
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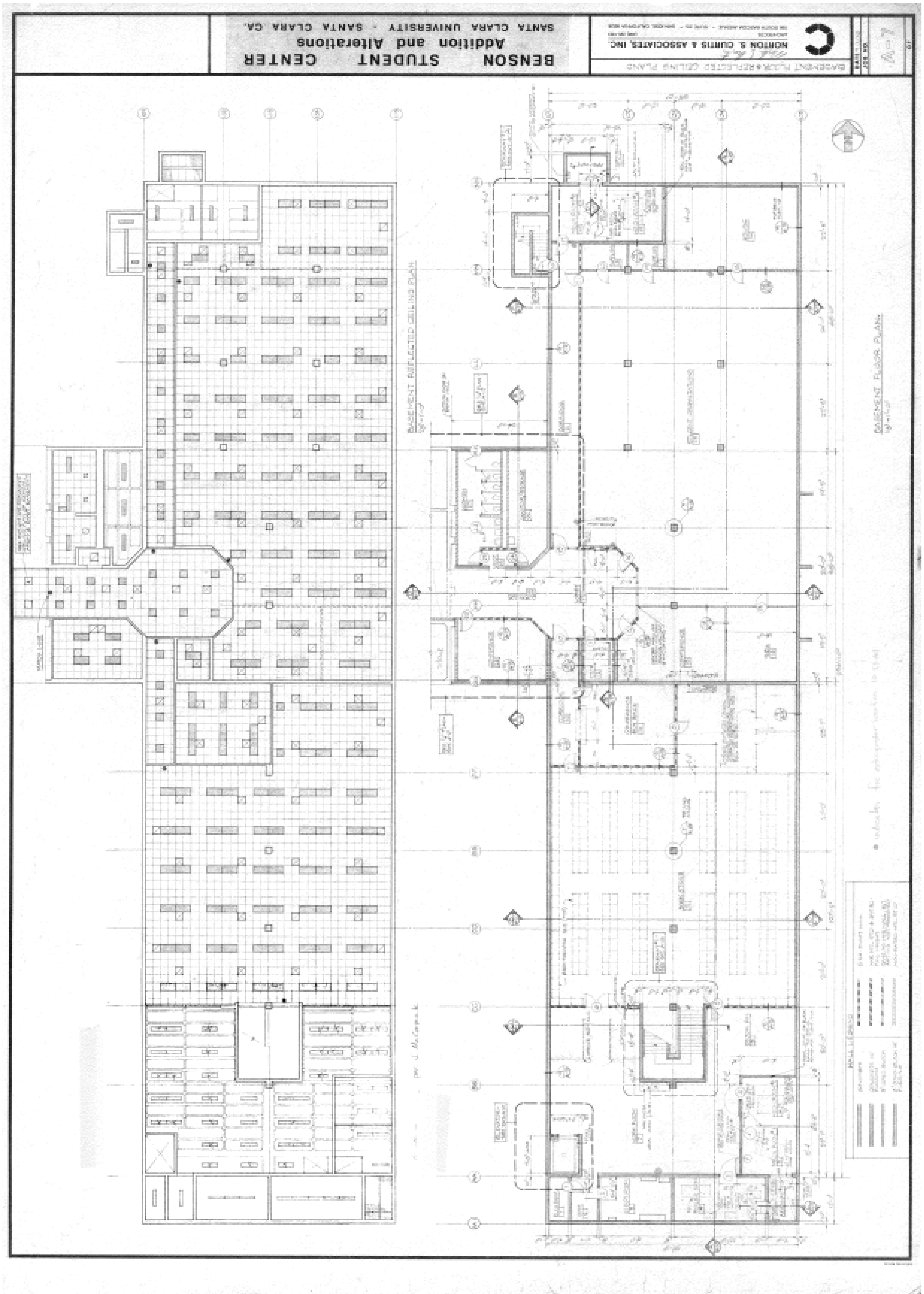
REVISED
FIRST FLOOR REFLECTED CEILING PLAN
FIRST FLOOR PLAN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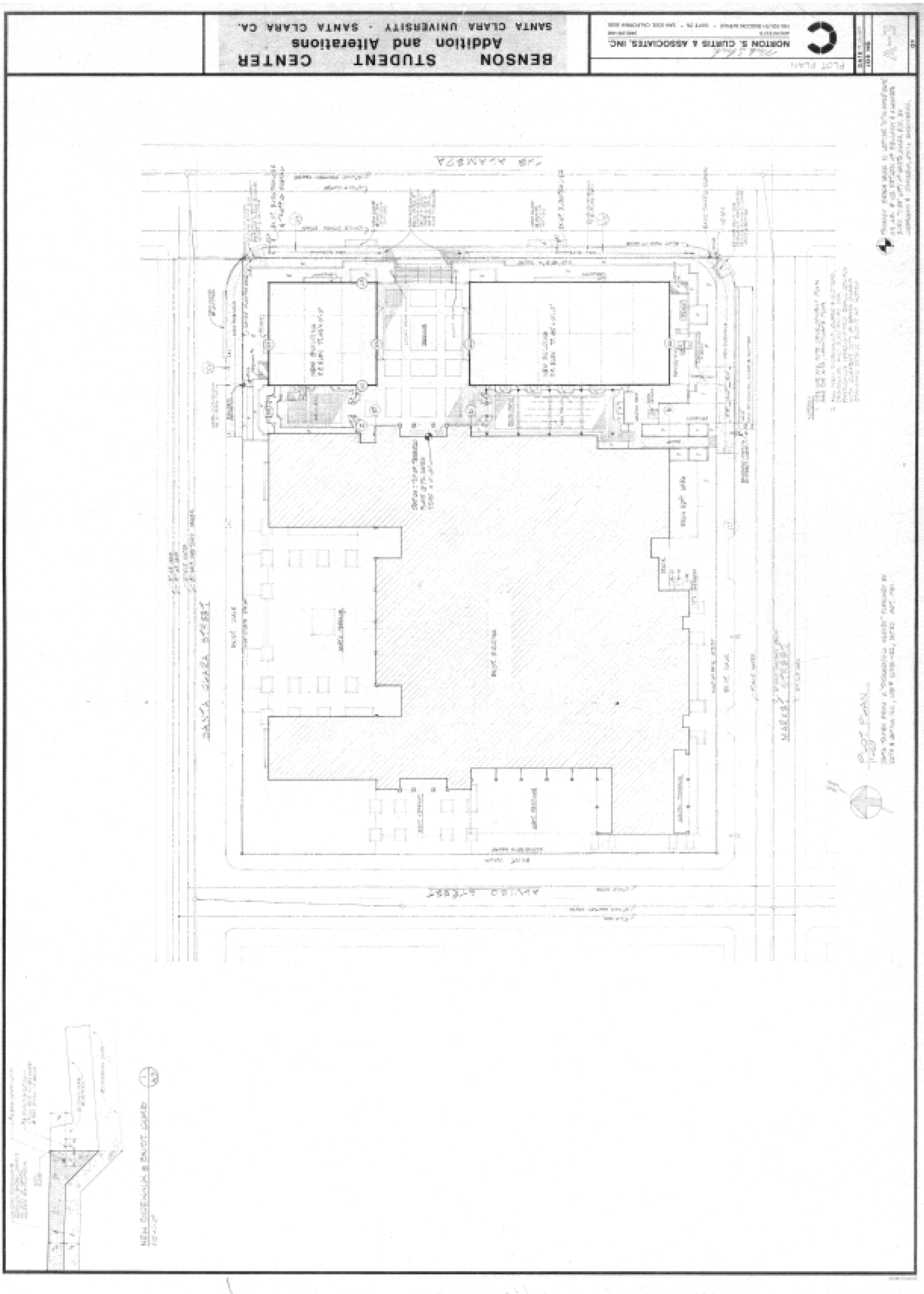
Addition and Alterations
SANTA CLARA UNIVERSITY · SANTA CLARA CA.
NORTON S. CURTIS & ASSOCIATES, INC.
BASEMENT REFLECTED CEILING PLAN
BASEMENT FLOOR PLAN
A-7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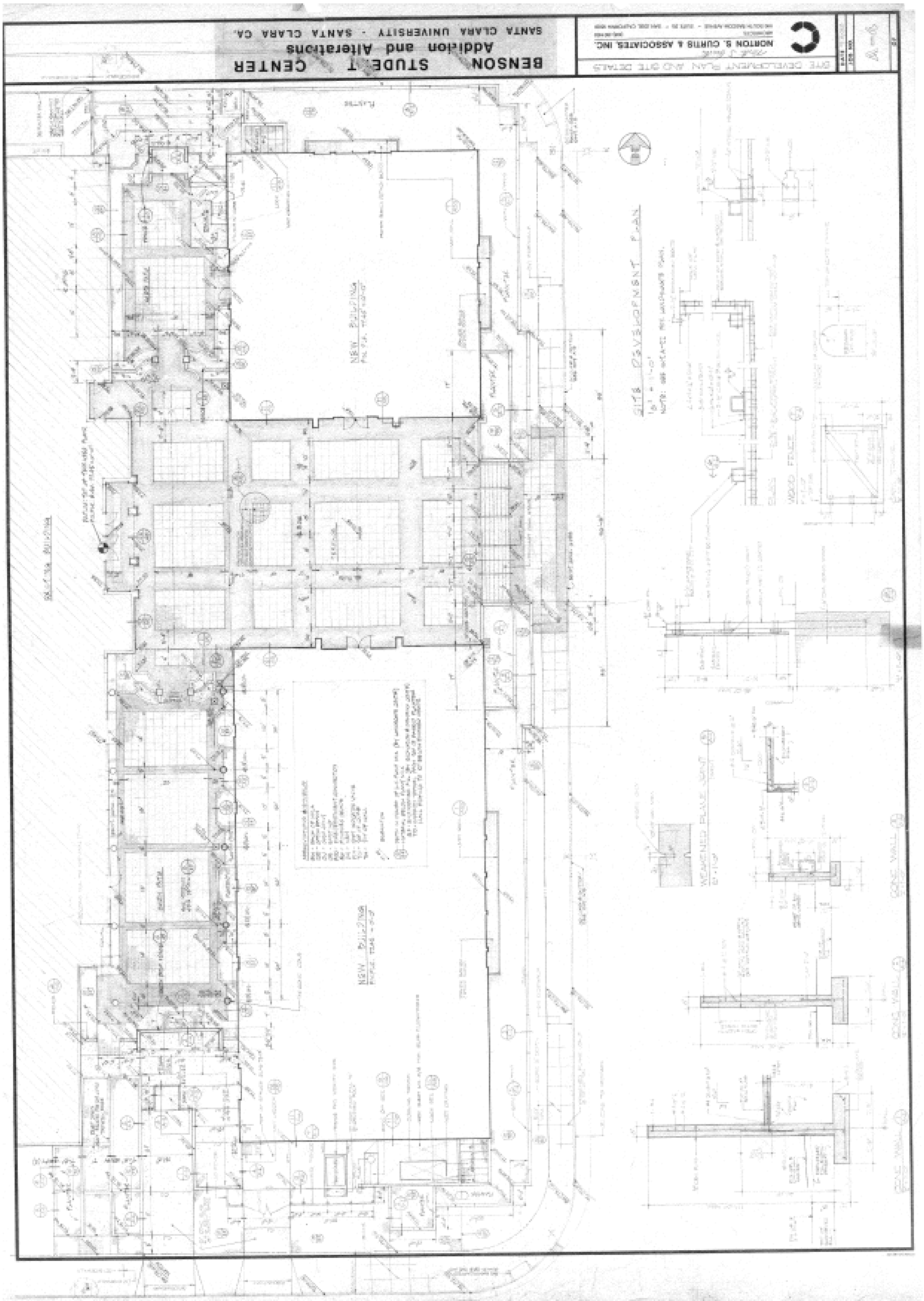
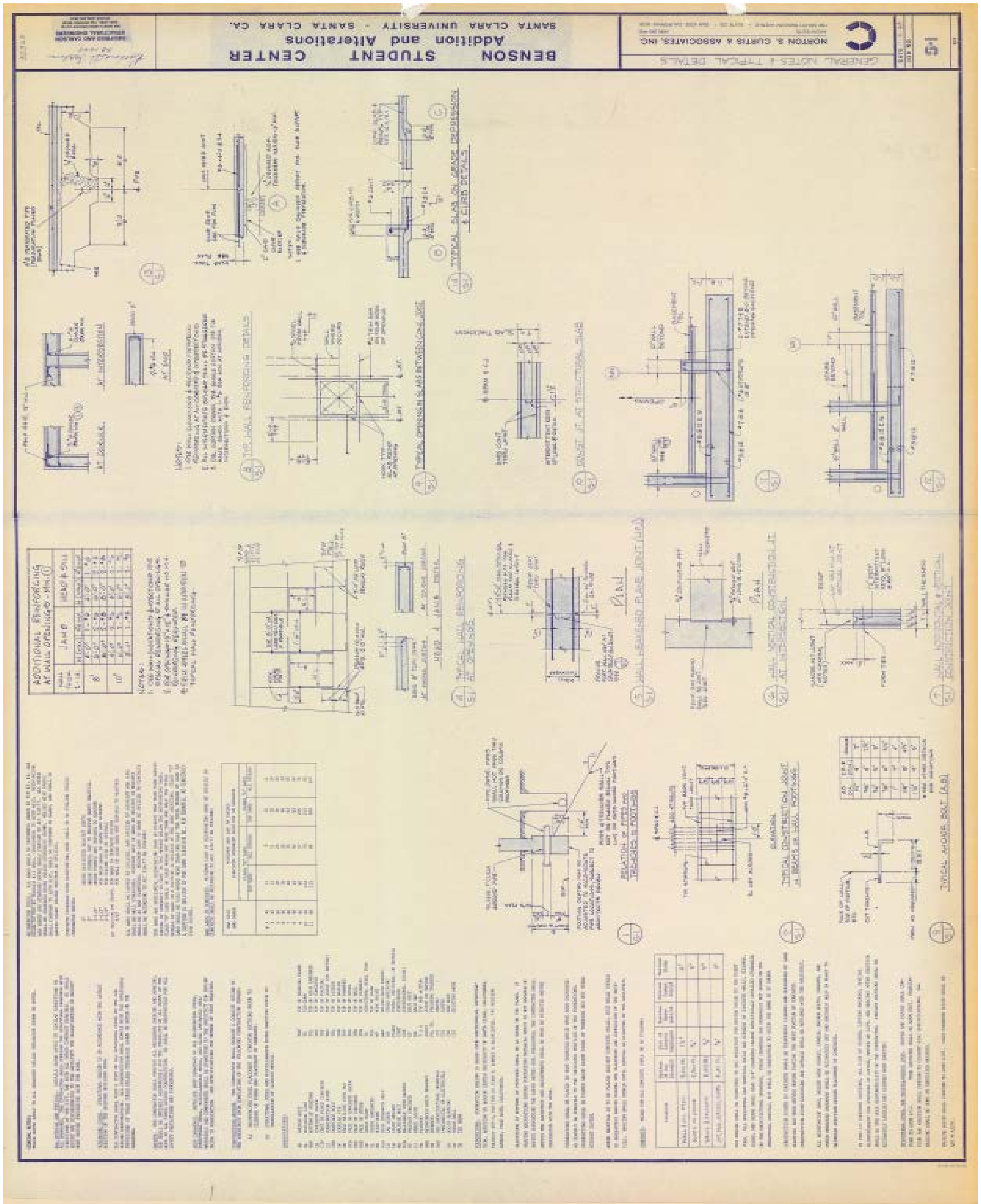
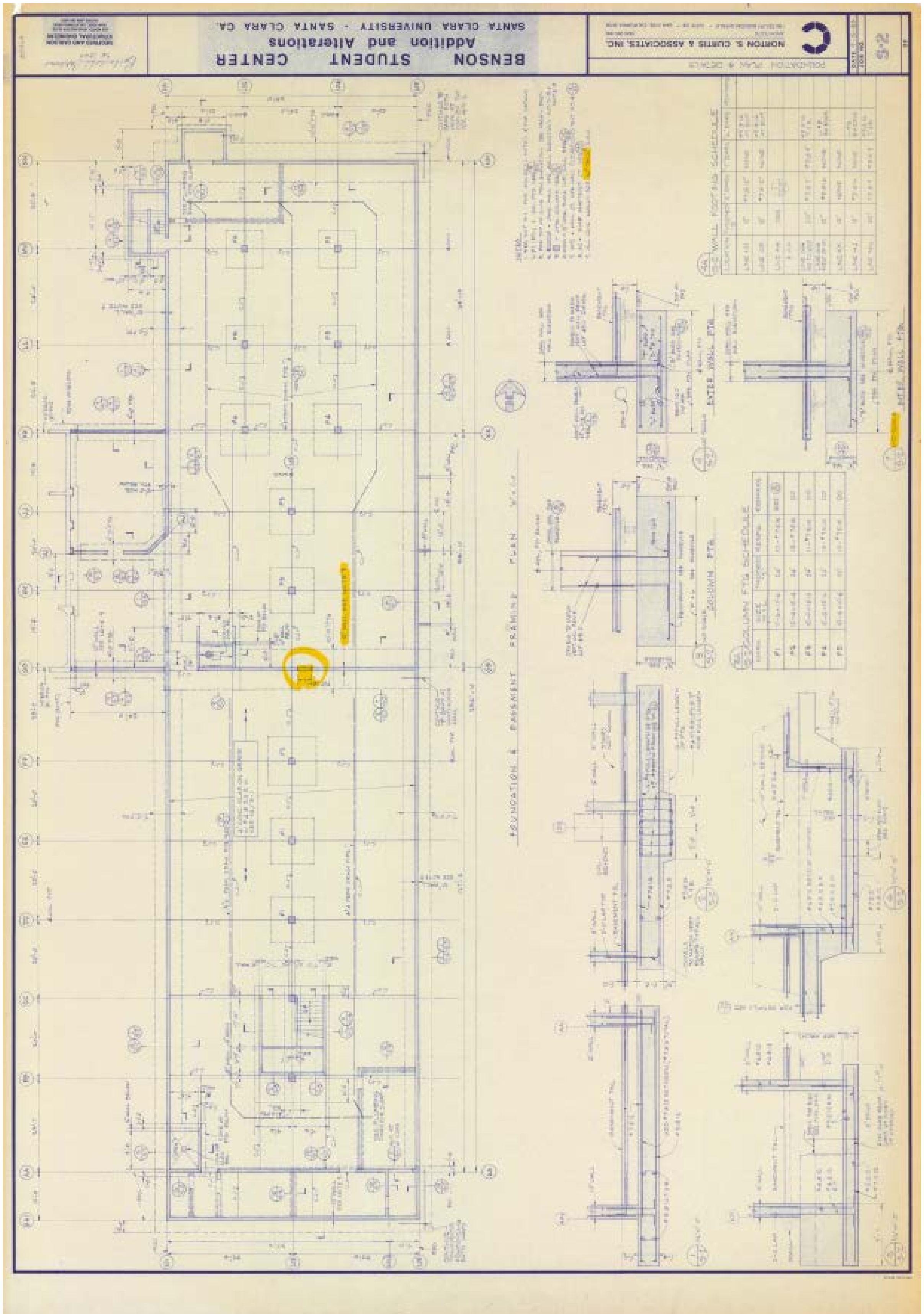
Addition and Alterations
SANTA CLARA UNIVERSITY - SANTA CLARA CA.
NORTON S. CURTIS & ASSOCIATES, INC.
PLOT PLAN
THE ALAMEDA
SANTA CLARA STREET
MARKET STREET


BENSON
CENTER
Addition and Alterations
SANTA CLARA UNIVERSITY - SANTA CLARA CA.
NORTON S. CURTIS & ASSOCIATES, INC.

Appendix B

The structural drawings of the original Bob Shapell Student Activities Hall, as provided by the Santa Clara Facilities Department

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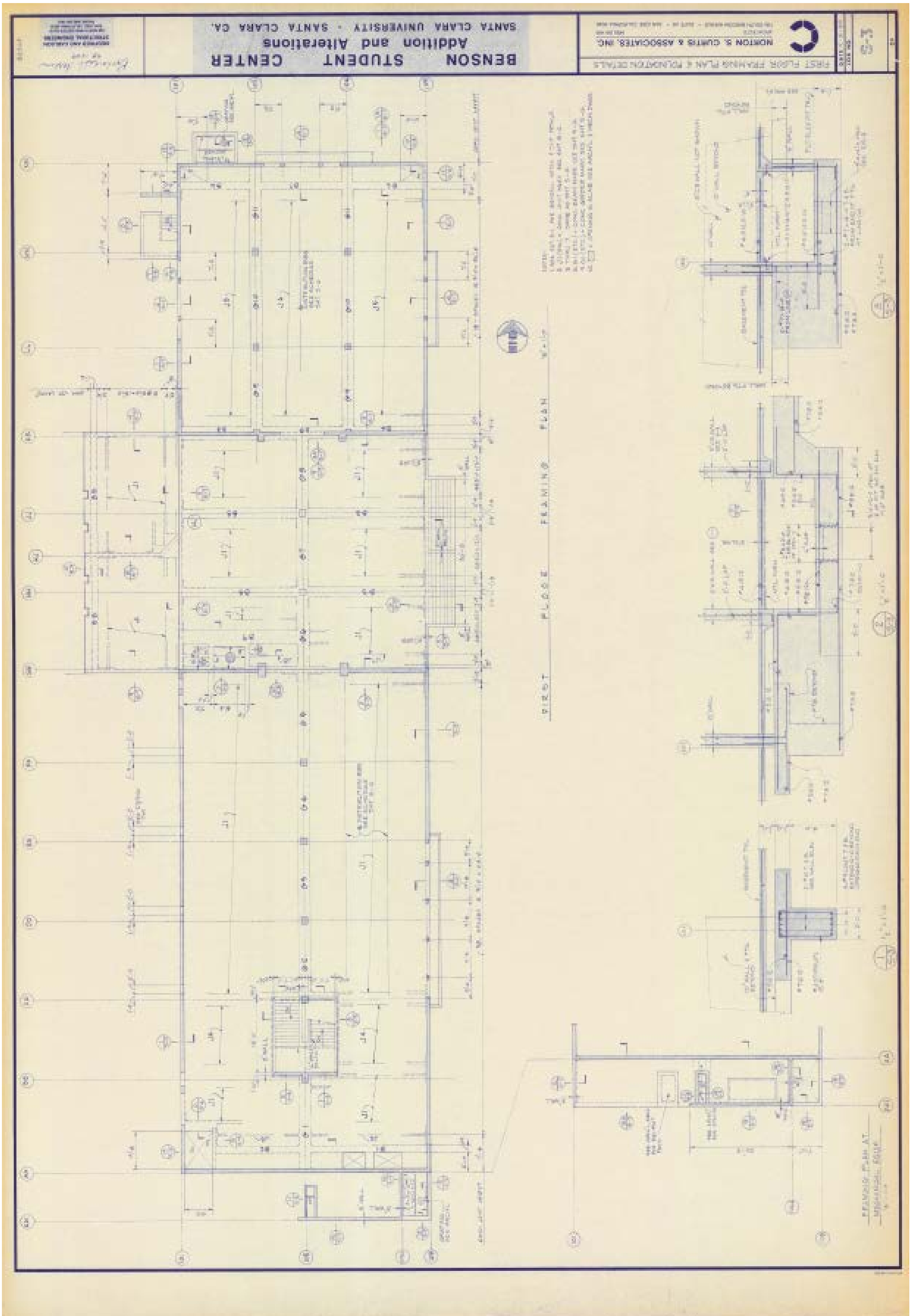
Addition and Alterations
SANTA CLARA UNIVERSITY - SANTA CLARA CA.
NORTON S. CURTIS & ASSOCIATES, INC.
S-2
FOUNDATION & BASEMENT FRAMING PLAN
WALL FOOTING SCHEDULE
COLUMN FTG. SCHEDULE
EXTER. WALL FTG.
INTER. WALL FTG.
COLUMN FTG.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Addition and Alterations
SANTA CLARA UNIVERSITY · SANTA CLARA CA.
NORTON S. CURTIS & ASSOCIATES, INC.
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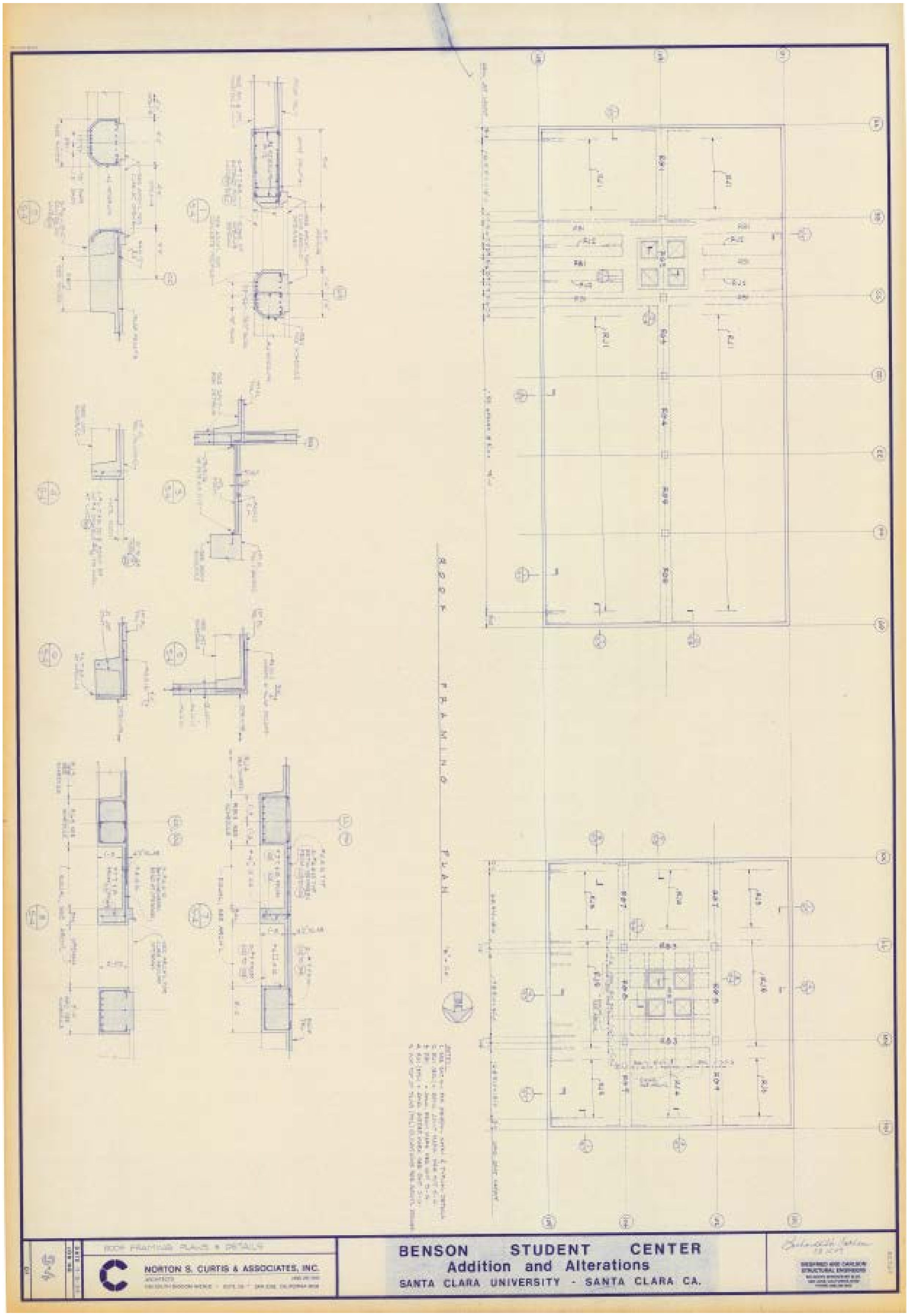
S-3
FIRST FLOOR FRAMING PLAN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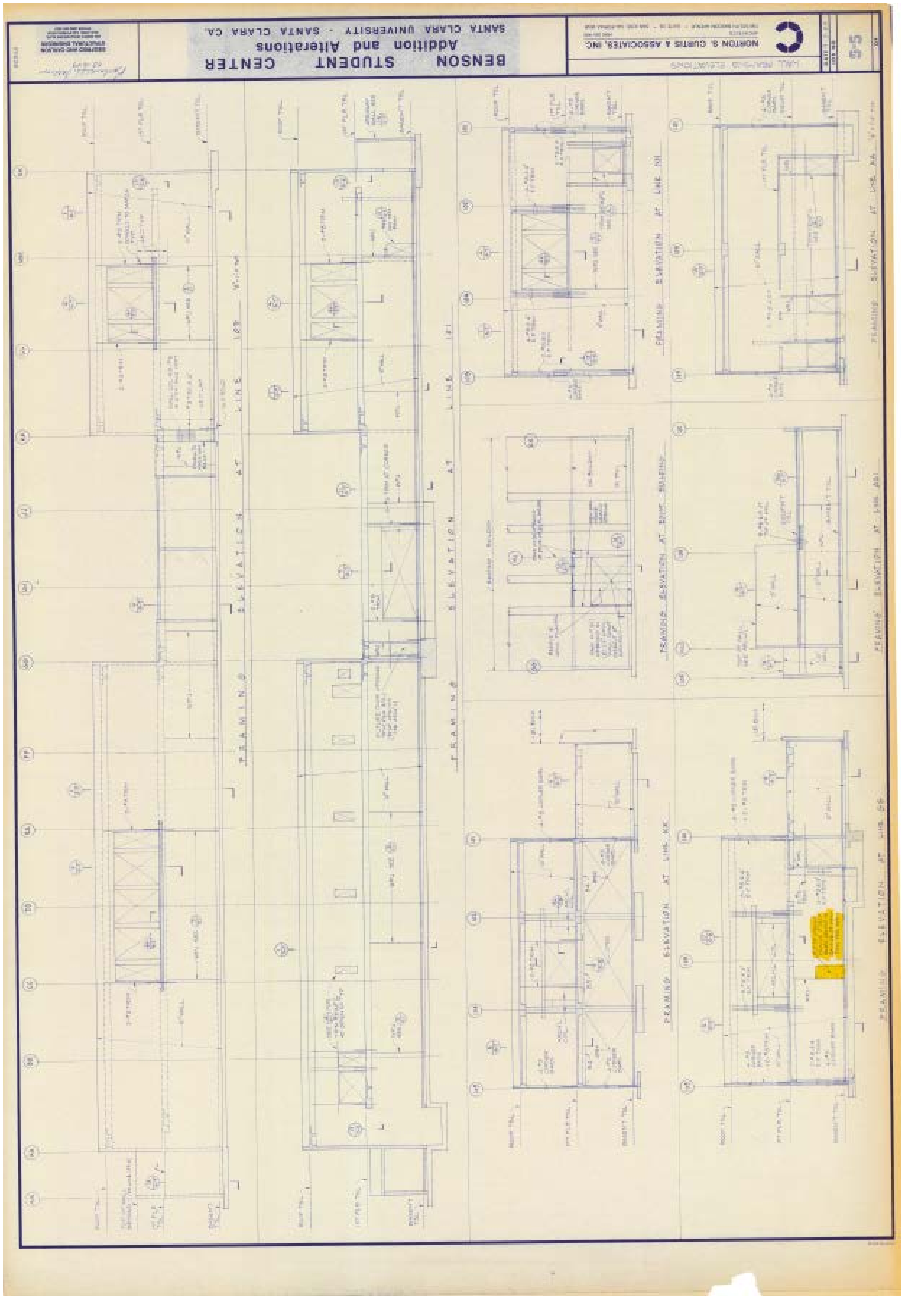
BENSON STUDENT CENTER
Addition and Alterations
SANTA CLARA UNIVERSITY - SANTA CLARA CA.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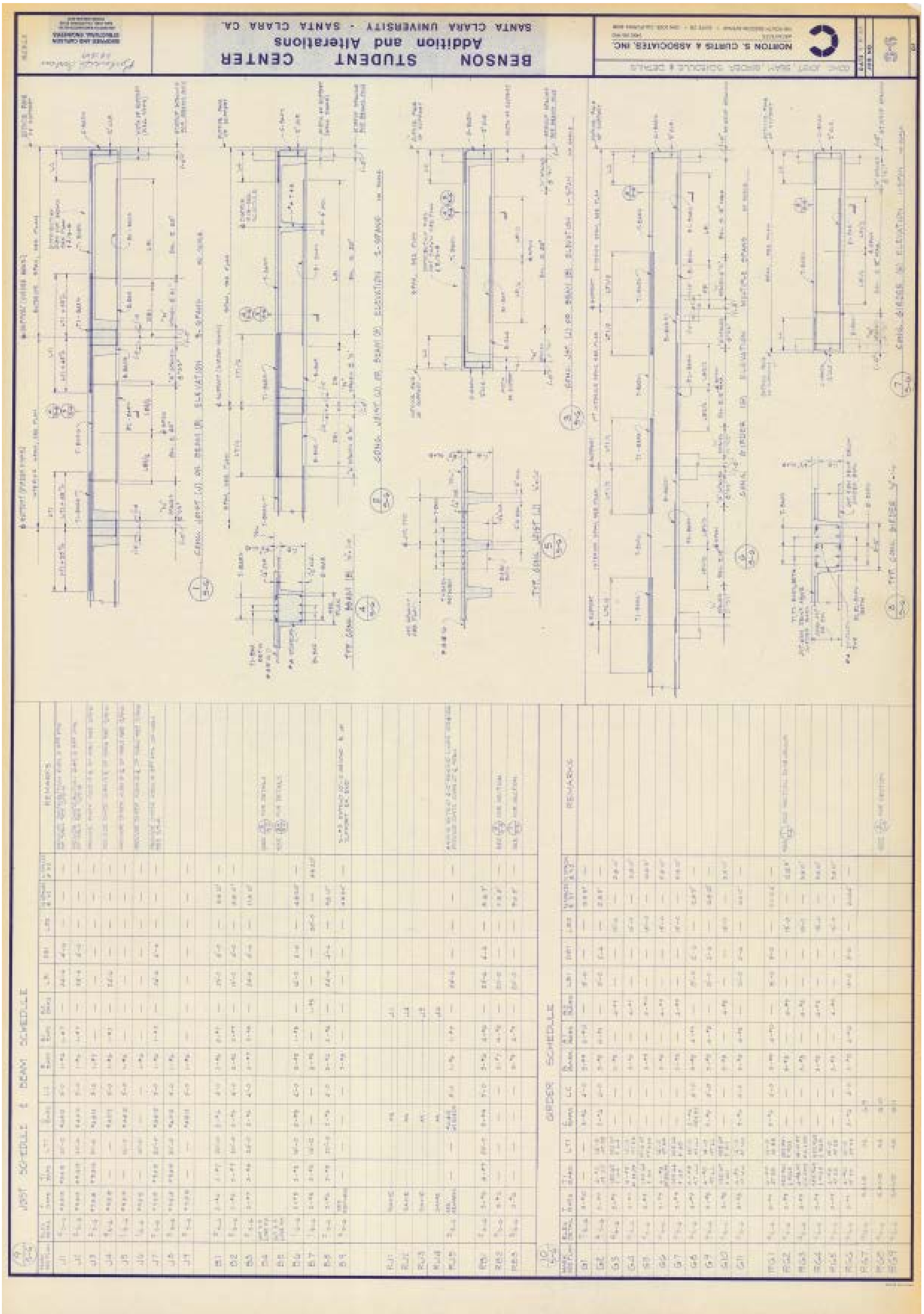
SANTA CLARA UNIVERSITY - SANTA CLARA CA.
NORTON S. CURTIS & ASSOCIATES, INC.
S-5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SANTA CLARA UNIVERSITY - SANTA CLARA CA.
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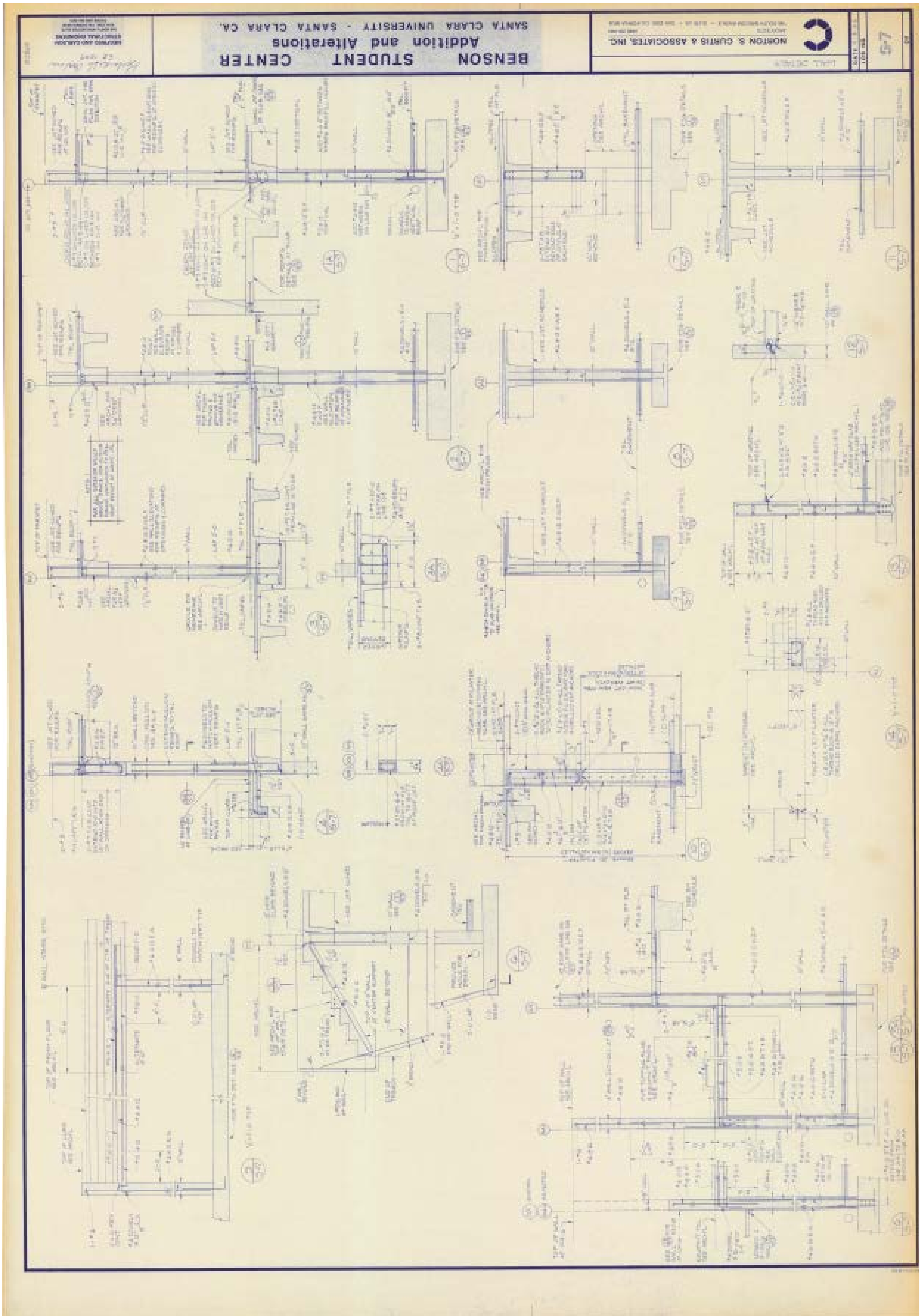
JOIST SCHEDULE & BEAM SCHEDULE
GIRDER SCHEDULE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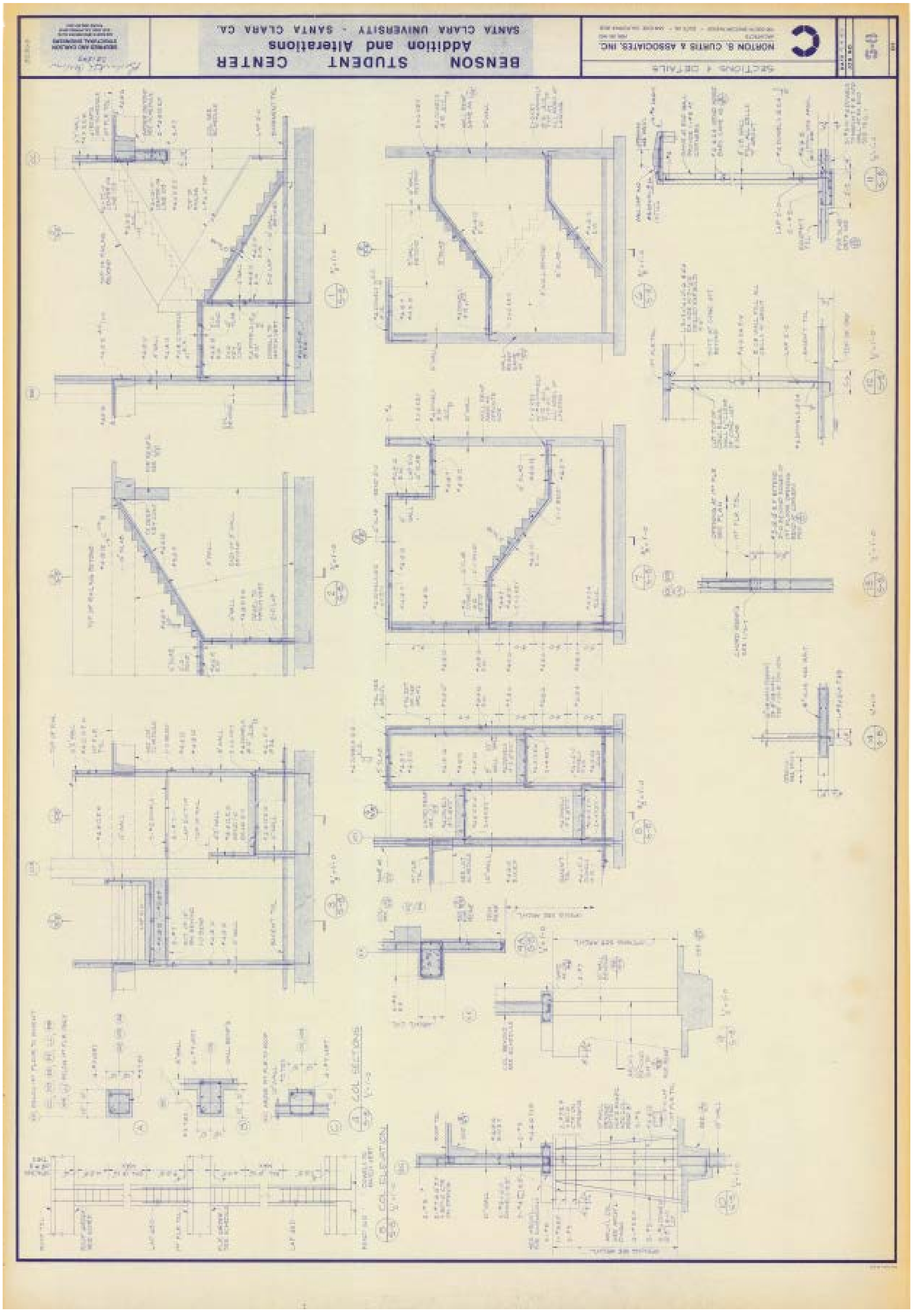
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NORTON S. CURTIS & ASSOCIATES, INC.
SECTIONS & DETAILS
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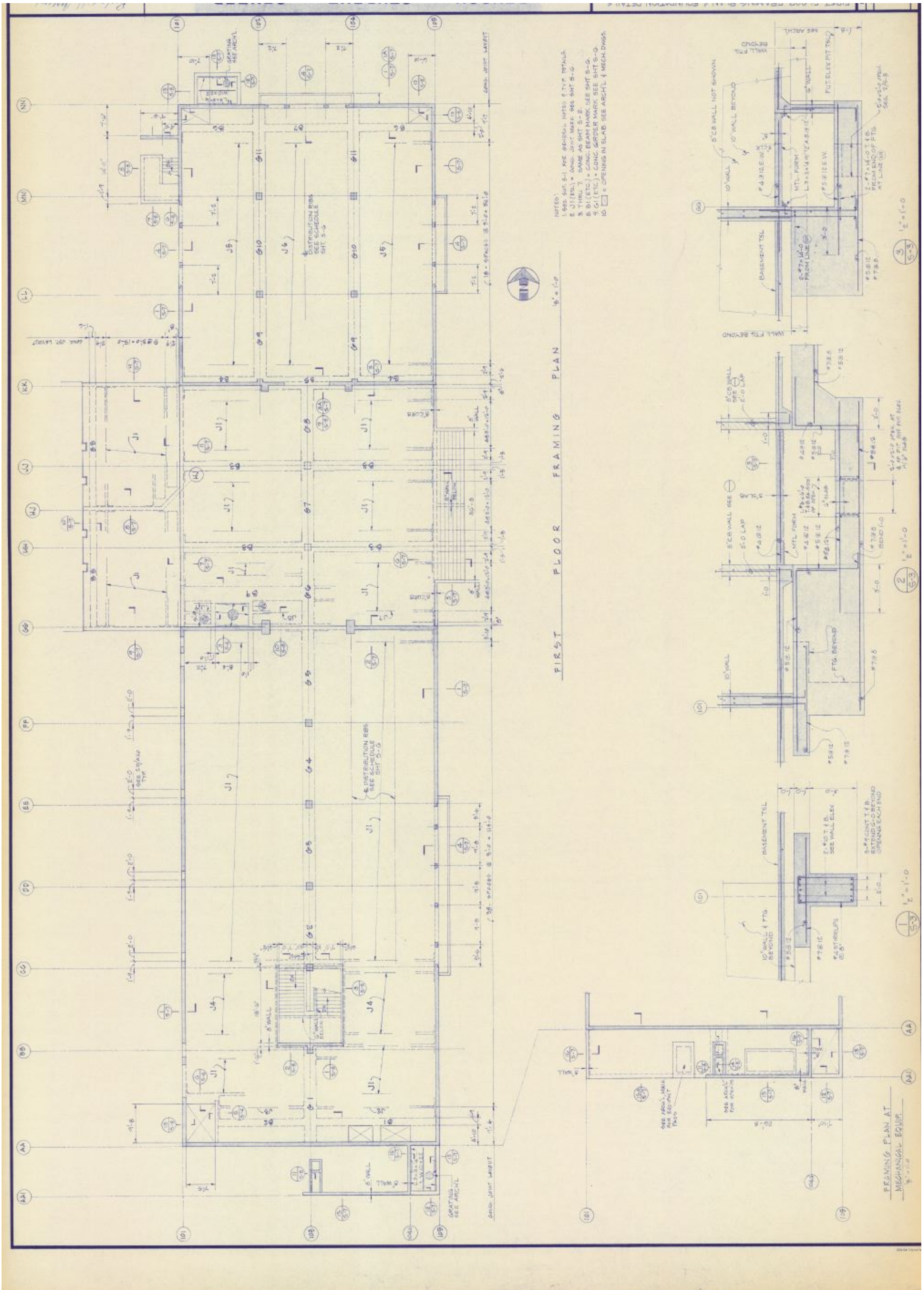
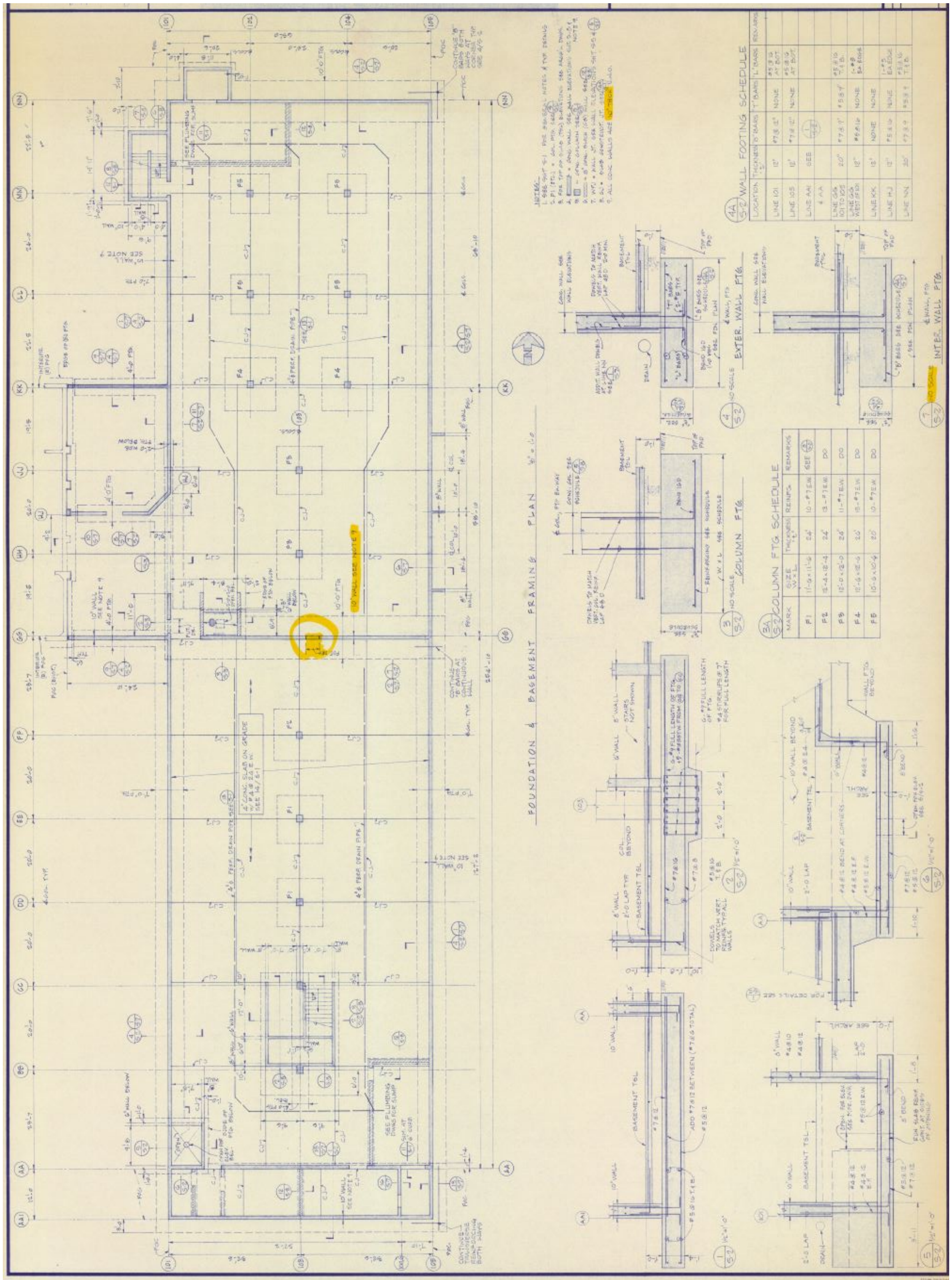

FIRST FLOOR FRAMING PLAN
FRAMING PLAN AT MECHANICAL EQUIP.


FOUNDATION & BASEMENT FRAMING PLAN
10' WALL SEE NOTE 7
COLUMN FTG SCHEDULE
WALL FOOTING SCHEDULE
EXTER. WALL FTG
INTER. WALL FTG
NO SCALE

Appendix C

The Benson Memorial Center Geotechnical Report as provided by the Santa Clara Facilities Department

John V. Lowney & Associates
Foundation / Soil / Geological Engineers

145 Addison Avenue, Palo Alto, California 94301 415/328-6920

April 5, 1983
401-2, PA 11108

Mr. Edmond D. Leys
Department of Architecture
and Construction
University of Santa Clara
Santa Clara, California 95053

RE: GEOTECHNICAL INVESTIGATION
ADDITION TO BENSON CENTER
UNIVERSITY OF SANTA CLARA
SANTA CLARA, CALIFORNIA

Attention: Mr. Donald C. Akerland

Gentlemen:

In accordance with your request, we have performed a geotechnical investigation for the above project. The accompanying report presents the results of our field investigation work, laboratory tests, and engineering analyses. The soil and foundation conditions are discussed and recommendations for the soil and foundation engineering aspects of the project are presented.

Our report features the following: 1) A geotechnical cross section through the site showing pertinent geotechnical data to permit you to visualize the subsurface conditions in profile. 2) A geologic hazards study done to reconnaissance level by our certified engineering geologist to provide evaluation of certain hazards such as faulting, liquefaction, and seismic differential compaction. 3) An executive summary on the following page to enable you to quickly obtain the principal findings, conclusions, and recommendations of the investigation.

We refer you to the text of the report for detailed recommendations. If you have any questions concerning our findings please call.

Very truly yours,

JOHN V. LOWNEY & ASSOCIATES

J.V. Lowney

John V. Lowney

JVL:DWB:lcp

UNIVERSITY OF SANTA CLARA
APR 7 1983
ARCHITECTURE & CONSTRUCTION

Copies: Addressee (4)
Norton S. Curtis, AIA & Associates (2)
Attention: Mr. Norton S. Curtis
Siegfried and Carlson (1)
Attention: Mr. Richard H. Carlson

GEOTECHNICAL INVESTIGATION

For

ADDITION TO BENSON CENTER
UNIVERSITY OF SANTA CLARA
SANTA CLARA, CALIFORNIA

To

University of Santa Clara
Department of Architecture
and Construction
Santa Clara, California 95053

April 1983

John V. Lowney & Associates

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continued

GEOTECHNICAL INVESTIGATION
ADDITION TO BENSON CENTER
UNIVERSITY OF SANTA CLARA
SANTA CLARA, CALIFORNIA

INTRODUCTION

In this report, we present the results of our geotechnical investigation for the proposed addition to Benson Center to be located fronting on the Alameda between Market Street and Santa Clara Street in San Jose, California. The purpose of this investigation was to evaluate the subsurface materials and conditions and provide recommendations concerning the soil and foundation engineering aspects of the project.

Project Description

The proposed addition to the Benson Center will be a one-story concrete building, approximately 255 by 65 feet in plan with a full basement and will be designed for a future second floor. A bookstore, offices, and lobby with connecting corridor are planned for the basement; a campus store, student lounge and more office space are planned for the first floor. Ultimate structural loads are expected to range up to 319 kips dead load plus live loads for interior columns, 10.1 to 14.1 kips per lineal foot for exterior wall loads and 13.6 kips per lineal foot for interior wall loads. A grid system of interconnected continuous footings is proposed.

SCOPE

The scope of work performed in this investigation included site reconnaissance, subsurface exploration, laboratory testing, engineering analysis of the field and laboratory data, and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing design criteria for site earthwork, building foundations, slab-on-grade floor, and basement walls. The scope of work was presented in detail in our agreement with you dated November 16, 1982.

This report has been prepared for the use of University of Santa Clara for application to the design of the proposed addition to Benson Center in accordance with generally accepted geotechnical engineering practices. No warranty is expressed or implied.

The investigation was conducted under the direction and review of John V. Lowney, Civil Engineer. Supervision of the subsurface exploration, laboratory testing, geotechnical engineering and the reconnaissance level geologic evaluation were performed by David W. Buckley, Engineering Geologist, Civil Engineer.

SITE INVESTIGATION

Exploration Program

A subsurface exploration and surface reconnaissance were performed on December 13, 1982 using a truck-mounted, continuous flight auger to explore and sample the subsurface soils. Four exploratory borings were drilled to depths ranging from 25 to 30 feet at the locations shown on the Site Plan, Figure 1. Logs of our borings and details regarding our field investigation are included in Appendix A; the results of our laboratory tests are discussed in Appendix B. A geotechnical cross section through the length of the building site, summarizing pertinent geotechnical data and permitting visualization of the subsurface conditions in profile is presented as Figure 2.

Surface

At the time of drilling, the surface was occupied by a parking lot pavement consisting of 2 to 3 inches of asphaltic concrete over 3 inches of aggregate base.

Subsurface

The soils encountered can be roughly grouped in the following three strata:

<u>Stratum A</u>: Dark gray, very stiff, silty clay (CL) from two to three feet thick. Penetration resistance values in this layer ranged from 26 to 43 blows per foot.

<u>Stratum B</u>: Light brown to gray-brown, firm to very stiff silty sandy clay (CL) with lenses of loose clayey sand (SC) and medium dense firm to medium grained sand with trace silt (SM) approximately 9 to 12 feet thick. Penetration resistance values ranged from 8 to 38 blows per foot. Torvane shear strength values ranged from 700 to 1800 pounds per square foot.

<u>Stratum C</u>: Blue-gray, firm to very stiff, silty clay (CL) with occasional lenses of sand with trace silt (SM) to the maximum depth explored of 30 feet. Penetration resistance values ranged from 13 to 30 blows per foot. Torvane shear strength values ranged from 800 to 2500 pounds per square foot.

Groundwater

Free groundwater was not encountered in the borings at the time of drilling. It should be pointed out, however, that none of the borings were left open for more than 15 minutes which may not have been sufficient time to permit groundwater to enter the borings in the relatively impermeable clays. Groundwater was also not encountered in the more permeable sand stratum found in Boring EB-1 to approximately elevation +56 feet. In addition, please note that fluctuations in the level of the groundwater may occur due to variations in rainfall and other factors at the time measurements were made. As a precautionary measure however, we recommend that a subdrain system be placed beneath the basement slab.

Seismicity

The San Francisco Bay Area is recognized by geologists and seismologists as one of the most active regions in the United States. The significant earthquakes which occur in the Bay Area are generally associated with crustal movement along well-defined, active fault zones. These zones include the San Andreas, Hayward and Calaveras Faults, located approximately 10.5 miles southwest and 7.5 and 10.0 miles northeast of the site, respectively.

Although research on earthquake prediction has greatly increased in recent years, seismologists cannot predict when and where an earthquake will occur. Nevertheless, on the basis of current technology, it is reasonable to assume that the proposed development will be subjected to at least one moderate to severe earthquake during the 50-year period following construction. During such an earthquake, the danger from fault offset on the site is slight, but strong shaking of the site is likely to occur.

Geologic Hazards

A brief qualitative evaluation of certain geologic hazards was made during this investigation resulting in the comments presented below:

1. Fault Rupture - No known active faults are believed to exist within the site, and no fault rupture is therefore anticipated.

2. Ground Shaking - During moderate to severe earthquakes occurring in the general region, strong ground shaking is expected to occur at the site as is typical for such sites in the San Francisco Bay Area.

3. Liquefaction - Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils that are most susceptible to liquefaction are clean, loose, saturated, uniformly graded, fine-grained sands. The soil encountered below the proposed basement level, 13 feet below the surface, Stratum C, consisted of firm to very stiff silty clay with occasional lenses of fine to medium-grained sand with a trace of silt. Free groundwater was not encountered. Consequently, in our opinion, the probability of liquefaction of the soil immediately underlying the building basement area is low.

4. Differential Compaction - The subsurface soils at the site described in Stratum C vary in composition both vertically and laterally over the site. During a major earthquake, differential compaction of this alluvial soil is possible. In our opinion, however, and based upon judgment, the probability of ground movement due to differential compaction at the site will probably be low in general during major earthquake shaking.

DESIGN RECOMMENDATIONS

Conclusions

From a geotechnical engineering standpoint, it is our opinion that the site is suitable for the proposed development provided the recommendations presented in this report are incorporated in the design and construction of the project. The primary geotechnical feature of concern is the compressible silty clay encountered beneath the basement footing level. An estimate of the total post-construction settlement under the presently planned foundation scheme is contained in a later section of the report.

Since subsurface conditions may vary considerably from those expected on the basis of relatively small diameter borings, and to assure that our report recommendations have been properly implemented, we recommend that we be retained to: 1) review the final construction plans and specifications, and 2) observe the earthwork and foundation installations.

EARTHWORK

Clearing and Site Preparation

The site should be cleared of all surface and subsurface deleterious materials including buried utility lines and pavements. Any resulting excavations that extend below the planned finish site grades should be cleaned and backfilled with suitable material compacted to the requirements given below under the section captioned "Compaction."

Subgrade Preparation

After the site has been properly cleared and the necessary excavations made, the exposed surface soils in those areas to receive structural fill, slabs-on-grade or pavements should be scarified to a depth of 6 inches, moisture conditioned to slightly above optimum moisture content, and compacted in accordance with the requirements for structural fill given below under the section captioned "Compaction."

Material for Fill

All on-site soils having an organic content of less than 3 percent by volume are suitable for use as fill at the site. In general, fill material should not contain rocks or lumps larger than 6 inches in greatest dimension with no more than 15 percent larger than 2.5 inches. Imported fill material should be predominantly granular with a sand equivalent of 10 or more (ASTM D 24-19, latest edition).

Compaction

All structural fill placed at the site and scarified surface soils in those areas to receive structural fill or slabs-on-grade should be compacted by mechanical means to at least 90 percent relative compaction as determined by ASTM Test Designation D 1557, latest edition. Fill should be placed in lifts not exceeding 8 inches in uncompacted thickness. The upper 6 inches of subgrade in pavement areas should be compacted to at least 95 percent relative compaction (ASTM D 1557, latest edition).

Trench Backfill

Pipeline trenches should be backfilled with compacted structural fill. If on-site soil is used, the material should be placed in lifts not exceeding 8 inches in uncompacted thickness and compacted to at least 85 percent relative compaction (ASTM D 1557, latest edition) by mechanical means only. Imported sand may also be used for backfilling trenches provided the sand is compacted to at least 90 percent relative compaction. In all pavement and building pad areas, the upper 3 feet of trench backfill should be compacted to at least 90 percent relative compaction for on-site soils, and to at least 95 percent where imported sand backfill is used. In addition, the upper 6 inches of all trench backfill in pavement areas should be compacted to at least 95 percent relative compaction.

Temporary Slopes

Temporary slopes will be necessary during the excavation for the basement. We recommend that unshored temporary excavations be sloped at an inclination no steeper than 1:1 (horizontal to vertical). Additonally, we recommend that the tops of these slopes be located at least 5 feet away from adjacent buildings. Because of the variable nature of the underlying soils, field modifications of temporary cut slopes will be required during construction if adverse conditions are exposed. Construction equipment and material stockpiles should be located more than 5 feet behind temporary construction slopes to avoid overstressing the temporary slope.

Surface Drainage

Positive surface gradients should be provided adjacent to the building to direct surface water away from the foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed adjacent to the structure or on the pavements.

Construction Observation

All grading and earthwork should be performed under the observation of our representative to see that proper site preparation, selection of satisfactory fill materials, as well as placement and compaction of the fills has been performed. Sufficient notification to us prior to earthwork is essential. All earthwork should be performed in accordance with the Guide Specifications - Site Earthwork presented in Appendix C. However, the guide specifications are only general in nature and the actual project specifications should incorporate all requirements contained in the text of this report.

Variations in soil conditions are possible and may be encountered during construction. In order to permit correlation between the soil data obtained during our field and laboratory investigations and the actual subsurface conditions encountered during construction and to observe conformance with the plans and specifications as originally contemplated, it is essential that we be retained to perform the required continuous or intermittent review during construction or the earthwork, excavation, and foundation phases.

FOUNDATIONS

Footings

We recommend that the proposed building be supported on conventional continuous spread footings bearing on either undisturbed natural soils or compacted fills. Any building foundations adjacent to utility trenches should have their bottom depths located below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the edge of the trench. Located at these depths, the footings may be designed for an allowable bearing pressure of 2000 pounds per square foot due to dead loads, and 2500 pounds per square foot due to dead plus live loads with a one-third increase for all loads including wind or seismic. All footings should have a minimum width of 12 inches. These allowable bearing pressures are net values; the weight of the footing can be neglected for design purposes. All footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the trench.

All continuous footings should be designed with adequate top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. It is essential that we inspect the footing excavations prior to placing reinforcing steel.

Post-construction total settlements under the proposed static loading conditions and foundation scheme are expected to range from approximately 1 inch at the building center to 1 3/4 inches at the building corners over the 30-year period following construction.

Slabs-On-Grade

Prior to final construction of the slab, the subgrade surface should be proof-rolled to provide a smooth, firm surface for slab support.

As a precautionary measure, we recommend a subdrain drainage system consisting of 4-inch diameter perforated drain-pipe and 14-inches of 1/2-inch crushed rock beneath the basement slab. A plan view of the recommended subdrain layout and a profile view of the recommended subdrain system are shown in Figure 3. The subdrains are also shown in the Site Plan, Figure 1.

To minimize vapor transmission we recommend that an impermeable vapor barrier be placed over the 1/2-inch crushed rock described above. The vapor barrier should be covered with a 2-inch sand buffer to protect it during construction. The sand should be lightly moistened just prior to placing the concrete.

Due to the moderate expansion potential of the clayey native subgrade, we recommend that the contractor take special measures to protect the subgrade from any inflow of water during construction, especially after the basement floor slab has been cast. Areas to receive special attention include slab joints and areas where building columns pass through the floor slab.

Basement Walls

The proposed basement walls should be designed to resist lateral earth pressure from adjoining natural materials and/or backfill as well as any surcharge loads. We recommend that these walls, which are restrained from movement at the top, be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot plus a uniform pressure of 8H pounds per square foot, where H is the distance in feet between the top of the footing and the top of the wall. Such walls should also be designed to resist an additional uniform pressure equivalent to one-half of any surcharge loads applied at the surface. The preceeding pressures assume sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level.

Adequate drainage may be provided by a subdrain system positioned behind the walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed down) and below the adjacent slab elevation. The pipe should be embedded in 12 inches of Class 2 Permeable Material (California Department of Transportation Standard Specifications, latest revision); the remaining backfill behind the wall should consist of 1/2-inch crushed rock or 3/8-inch pea gravel extending at least 2 feet out from the wall and within 2 feet of the level of the outside finish grade. The upper 2 feet should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump. Walls should be damp-proofed.

Retaining walls should be supported on spread footing foundations designed in accordance with the recommendations presented previously under "Footings."

Lateral load resistance for the walls can be developed in accordance with the recommendations presented immediately below under "Lateral Loads."

Cantilever Retaining Walls

The proposed unrestrained retaining walls in landscaped areas should be designed to resist an equivalent fluid pressure of 35 pounds per cubic foot if the on-site clayey material is used for backfill and 30 pounds per cubic foot if granular imported fill material is used.

The preceeding pressures assume sufficient drainage behind the walls to prevent the build up of hydrostatic pressures from surface water infiltration and/or a rise in the groundwater level. Adequate drainage may be provided by weep holes with permeable material installed behind the walls or by means of a system of subdrains.

Lateral Loads

Lateral loads may be resisted by friction between the footings and the supporting subgrade. A friction resistance against sliding equal to 300 pounds per square foot plus (0.20 x actual dead load pressure) can be assumed to act over any footing base surface. In addition to the above, lateral resistance may be provided by passive pressures acting against foundations poured neat in the firm un-formed footing excavations. We recommend that an allowable uniform passive pressure of 800 pounds per square foot be used in design.

* * * * *

EXECUTIVE SUMMARY

John V. Lowney & Associates has been retained by the University of Santa Clara to perform a geotechnical investigation for the proposed addition to the Benson Center to be located in Santa Clara, California. The purpose of the investigation has been to assess the subsurface conditions in the project area and to provide recommendations concerning the geotechnical engineering aspects of the project. Principal results, conclusions, and recommendations from the investigation are the following. Please note that this summary is not intended to be used for design purposes, as it is simply a synopsis of the major points of our report. Please see the text of the report for complete design recommendations.

1. The soils below the basement footing level consist primarily of firm to very stiff silty, sandy clay with lenses of loose, clayey sand; see the Geotechnical Cross Section, Figure 2. In our opinion, this soil will provide adequate bearing for shallow spread footing foundations.

2. Groundwater was not encountered during the drilling operations to the maximum depth explored of 30 feet. However, none of the borings were left open for more than 15 minutes, which may not have been sufficient time to permit groundwater to enter the borings in the relatively impermeable clays.

    a. As a precautionary measure, we therefore recommend a subdrain drainage system consisting of 4-inch diameter perforated drainpipe and 14-inches of 1/2-inch crushed rock beneath the basement slab. A plan view of the recommended subdrain layout and a profile view of the recommended subdrain system are shown in Figure 3. The subdrains are also shown in the Site Plan, Figure 1.

    b. To minimize vapor transmission we recommend that an impermeable vapor barrier be placed over the 1/2-inch crushed rock described above. The vapor barrier should be covered with a 2-inch sand buffer to protect it during construction. The sand should be lightly moistened just prior to placing the concrete.

3. The site is likely to experience strong seismic shaking during a moderate to severe earthquake which is expected to affect the San Francisco Bay Area during the 50-year period following construction. The probability of damage to the building from other geologic hazards, namely fault rupture, liquefaction, and differential compaction, is low.

4. The buildings may be supported on conventional, continuous, and/or isolated spread footings bearing on native soil or compacted structural fill. All footings can be designed for 2000 pounds per square foot dead loads, and 2500 pounds per square foot dead plus live loads, with a one-third increase for all loads including wind or seismic.

    a. Post-construction total settlements under the proposed static loading conditions are expected to range from approximately 1 inch at the building center to 1 3/4 inches at the building corners over the 30-year period following construction.

Appendix D

Hand calculations for the structural design system

Senior Design | Bob Shapell Student Activities Hall - Roof

ROOF LOADS

Length : 88 ft
Width : 69 ft

Live load : 20 psf (CBC 1607.1)

Dead Loads :

Roof components	Loads (psf)
Mechanical / Electrical	8
Fireproofing	2
Metal Deck	2.4
5-ply gravel	6.5
Suspended ceiling	2
insulation	2
	Σ : 22.9 psf

Metal Deck Trial 1 : PLN3

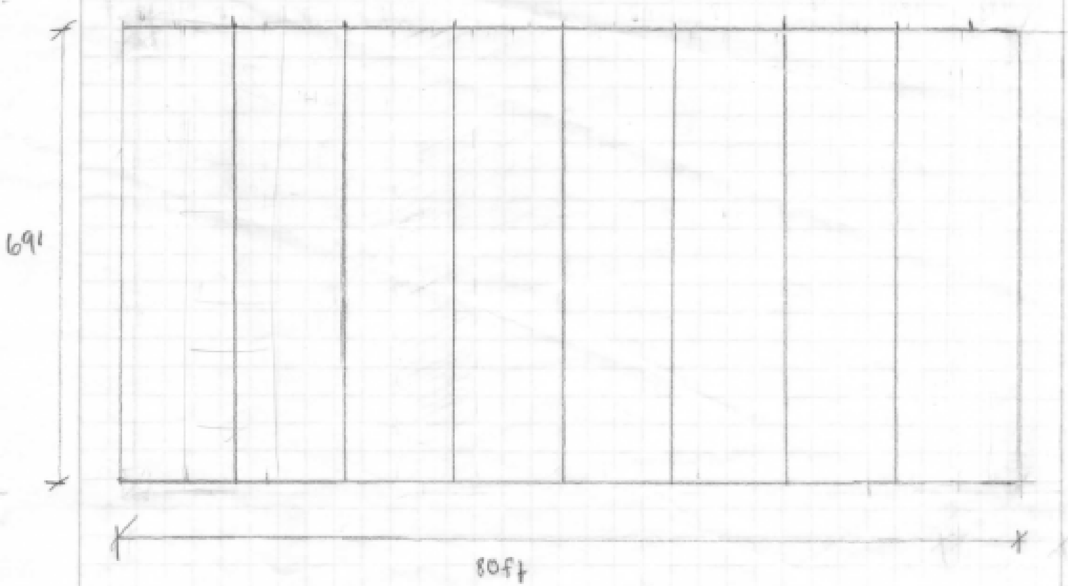
Deck gage : 20

Steel thickness : 0.375 in

ASTM A992

Span : 10 ft

$\sigma_{allowable}$ = 90 psf



Factored Loads

$P_u = 1.2D_L + 1.6L_L$ (CBC 1607.1)

$= 1.2(22.9\,psf) + 1.6(20\,psf)$

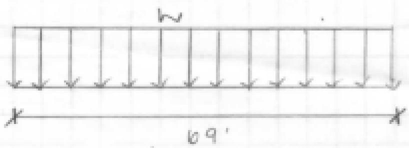
$P_u = 59.48$

$P_{service} = D_L + L_L$

$= 22.9\,psf + 20\,psf$

$= 42.9$

Interior Truss Joist - Uniform Load



Tributary width = 10'

$W_u = 10.0' \times 59.5\,psf = 595\,lb/ft$

$W_{D+L} = 10.0' \times 42.9\,psf = 429\,lb/ft$

$W_L = 10.0 \times 20\,psf = 200\,lb/ft$

$V_D = \frac{W_D L}{2}$

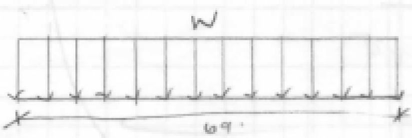
$V_u = \frac{W_u L}{2} = \frac{595\,lb/ft\,(60')}{2} = 17.85\text{ KIPS}$

Select truss joist 52DLH11 - New Millenium Building Systems

Approx weight = 39 lb/ft × 69' = 2691 lb

Max Load = 993 lb/ft > Wu ✓ safe load = 66540 lb

Exterior Truss Joist



Tributary width = 5.5'

$W_u = 5.0 \times 59.5\,psf = 298\,lb/ft$

$W_{D+L} = 5.0 \times 42.9\,psf = 216\,lb/ft$

$W_L = 5.0 \times 20\,psf = 100\,lb/ft$

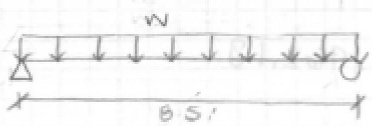
$V_u = \frac{W_u L}{2} = \frac{298\,lb/ft\,(60')}{2}$ = KIPS

Select truss joist 60DLH12

$W_D = \frac{W_D L}{2}$ $V_L = \frac{W_L L}{2}$

- Cantilever - check

Most Economical Beam For Angled Roof



$W_D = 22.9$ psf

$W_L = 20$ psf

Roof Loads

$U = 1.2D + 1.6L$

$P_u = 1.2(22.9 \text{ psf}) + 1.6(20 \text{ psf}) = 59.5$ psf

$P_{service} = D + L = 42.9$ psf

BEAMS

Tributary width = 10.0'

$W_u = 10' \times 59.5 \text{ psf} = 595 \text{ lb/ft}$

$W_{D+L} = 10' \times 42.9 \text{ psf} = 429 \text{ lb/ft}$

$W_L = 10 \times 20 \text{ psf} = 200 \text{ lb/ft}$

Moment Demand

$$M_u = \frac{W_u L^2}{8} = \frac{595 (8.5')^2}{8} \times \frac{12 \text{ in/ft}}{1000 \text{ lb/kip}} = \begin{array}{l} 5.38 \text{ k}\cdot\text{ft} \\ 64.5 \text{ k}\cdot\text{in} \end{array}$$

$$Z_x \text{ req'd} = \frac{M_u}{\phi F_y} = \frac{64.5 \text{ k}\cdot\text{in}}{(0.9)(50 \text{ ksi})} = 1.43 \text{ in}^3$$

Deflection Limits

$$\Delta_{LL} = \frac{L}{360} = \frac{8.5' \times 12\text{in}/1\text{ft}}{360} = 0.283 \text{ in}$$

$$\Delta_{max} = \frac{5 W_L L^4}{384 EI}$$

$$I_{min} = \frac{5 W_L L^4}{384 E \Delta_{max}} = \frac{5(0.2 \text{ lb/f})(8.5')^4 (12''/1')^3}{384 (29{,}000 \text{ ksi})(0.283 \text{ in})} = 2.86 \text{ in}^4$$

$$\Delta_{D+L} = \frac{L}{240} = \frac{8.5' \times 12 \text{in}/1\text{ft}}{240} = 0.425 \text{ in}$$

$$I_{min} = \frac{5 W_{D+L} L^4}{384 E \Delta_{max}} = \frac{5(0.429)(8.5')^4 (12''/1')^3}{384 (29{,}000 \text{ ksi})(0.425 \text{ in})} = 4.09 \text{ in}^4$$

← governs

Beam selection

$Z_x \geq 1.43\ in^3$
$I_x \geq 4.09\ in^4$ } Use W8x10 ; $Z_x = 8.87\ in^3$; $I_x = 30.8$

Moment capacity

W8x10 has non-compact flanges
Check strength per AISC Table 3-2
$\varnothing M_p = 32.9\ K \cdot ft > M_u = 5.38\ K \cdot ft$ ✓ok

Shear capacity

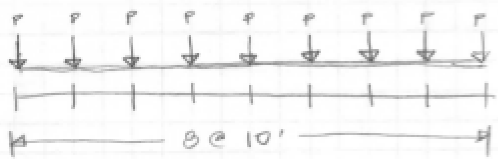
$\varnothing V_n = \varnothing\, 0.6 F_y A_W C_v$
~ Per AISC G2.1b ; $\varnothing = 1.0$, $C_v = 1.0$
~ AISC Table 1-1 ; $d = 7.89"$; $t_w = 0.170"$
$\varnothing V_n = (1.0)(0.6)(50 ksi)(7.89" \times 0.170")(1.0) = 40.24\ K$

$$V_u = \frac{W_u L}{2} = \frac{0.595\,(8.5)}{2} = 2.53\ \text{KIPS}$$

$\varnothing V_n = 40.24\ K > V_u = 2.53\ K$ ✓

Girder G1

Reaction from Each Beam



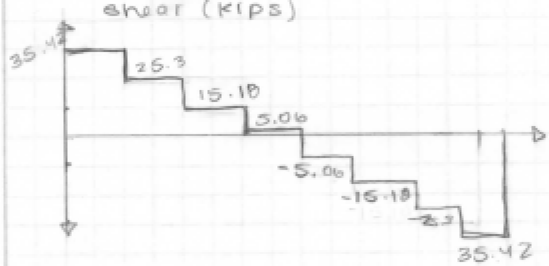
$P_u = \frac{W_u L}{2} = \frac{(0.595\ k/ft)(8.5)}{2} = 10.12\ k$

P_{D+L} : $(0.429\ k/ft)(8.5\ ft) = 3.65\ k$

$P_L = (0.2\ k/ft)(8.5\ ft) = 1.7\ k$

Reactions = 45.54 k

Moment demand



Moment (k·ft)

$M_u = 809.6\ k \cdot ft \times 12\ in/ft = 9715.2\ kip \cdot in$

$Z_{x\,req'd} = \frac{M_u}{\phi F_y} = \frac{9715.2\ k \cdot in}{(0.9)(50\ ksi)} = 215.89\ in^3$

Girder:

Try: W24 x 84 : $\phi M_p = 840\ k \cdot ft$

$I_x = 2370\ in^4$

$L_p = 6.89\ ft$

$L_r = 20.3\ f$

$L_b = 10.0\ ft < L_r$, ∴ No Lateral Torsional Buckling

COLUMN DESIGN

Soil Bearing Pressure

DL = 2000 psf

DL + LL = 2500 psf

Axial load = 36.25 kips = ϕP_n

Concrete Encasing

$f'_c = 3$ ksi

$$A_{g\,min} = \frac{P_n}{0.45 f'_c} = \frac{45.45\ k}{0.45(3\,ksi)} = 33.67\ in^2 = \pi/4\, d^2$$

$$d_{min} = \sqrt{\frac{33.67\,(4)}{\pi}} = 5.80\ in$$

Column minimum diameter = 6 in

Design: 24 in diameter > 6 in ✓

~ More physical stability

Area of column = $\pi/4\,(2.0\ ft)^2$

$= 3.14\ ft^2$

Height of column = 15 ft = K_L

Concrete

Volume = $3.14\ ft^2 \times 15\ ft = 47.12\ ft^3$

Weight = $47.12\ ft^3 \times 150$ pcf = 7068.6 lb

FOOTING DESIGN

column load = 7.07 kips

Truss load = + 36.25 kips

43.32 kips

Footing Area

$$45.28\ k \times \frac{1000\ lb}{k} \times \frac{ft^2}{2500\ lb} = 18.1\ ft^2 \rightarrow 20.25\ ft^2$$

∴ Footing = 4.5 x 4.5

Footing Thickness ≈ 3 ft

Column - Concrete Quantity

Area = 3.14 ft^2

Height = 15 ft

Volume = $15 \text{ ft} (3.14 \text{ ft}^2) \left(\frac{1 \text{ yd}^3}{27 \text{ ft}^3} \right) =$

$= 1.74 \text{ yd}^3$

Volume x 4 columns = 6.97 → 7.0 yd^3 concrete

Column Footing Concrete Quantity

Volume = $4.5' (4.5') (3')$

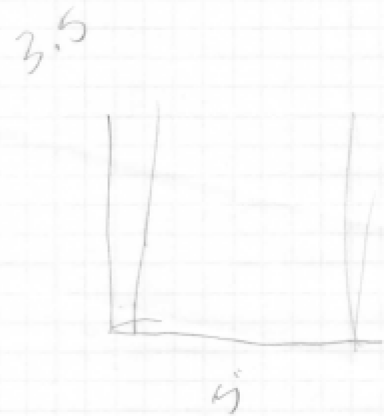
$= 60.75 \text{ ft}^3 \left(1 \text{ yd}^3 / 27 \text{ ft}^3 \right) = 2.25 \text{ yd}^3$

Volume x 4 footings = 9 yd^3 concrete

Gravel for Roof

Load = 6.5 psf

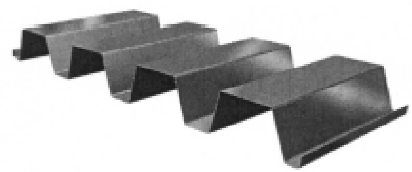
Weight = $6.5 \text{ psf} \times 5520 \text{ sf} \times \frac{1 \text{ ton}}{2000 \text{ lb}} = 17.94 \text{ ton}$



Appendix E

Information regarding member sizes selected for the proposed structural design system

Type PLN3™ or HSN3™

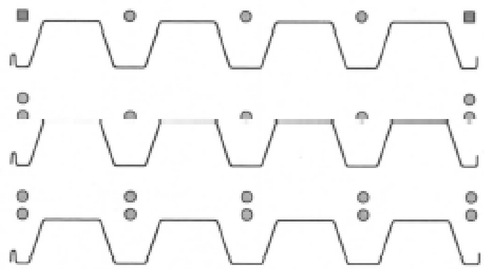


Attachment Patterns to Supports

32/5

32/7

32/10



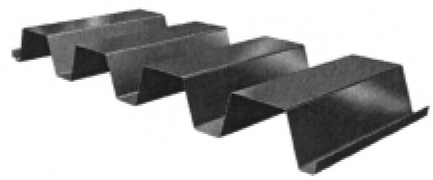
Note: ● indicates location of arc spot weld, power actuated fastener, or screw as indicated in the load tables.
■ indicates location of arc seam weld, power actuated fastener, or screw as indicated in the load tables.

Footnotes for Allowable Uniform Load Tables

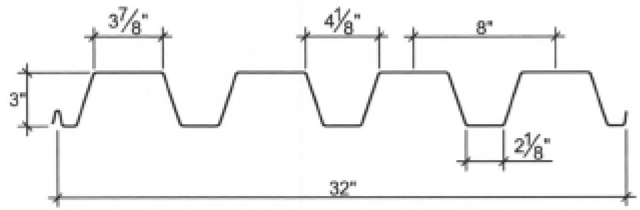
1. Stress = Allowable uniform load based on maximum allowable flexural stress in deck.
2. L/360, L240 or L/180 = Uniform load which produces selected deflection in deck.
3. The symbol ♦♦♦ indicates allowable uniform load based on deflection exceeds allowable uniform load based on stress.
4. Nominal uniform loads governed by stress may be determined by multiplying the allowable values in the table by $\Omega_b = 1.67$. LRFD loads may be determined by multiplying nominal loads by $\Phi_b = 0.95$.

Type PLN3™ or HSN3™

- 3" Deep Roof Deck
- Primer Painted or Galvanized
- PLN3 Deck used with PunchLok II System
- HSN3 Deck used with TSWs, BPs or Screws



Dimensions



PLN3 or HSN3 | HSN3-SS | HSN3-NS

Standard Interlocking Sidelap | Screw Fastened Sidelap | Nested Sidelap

Deck Weight and Section Properties

Gage	Weight		I_d for Deflection		Moment		Allowable Reactions per ft of Width (lb) — One Flange Loading					Allowable Reactions per ft of Width (lb) — Two Flange Loading				
	Galv	Painted	Single Span	Multi Span	$+S_{eff}$	$-S_{eff}$	End Bearing Length			Interior Bearing Length		End Bearing Length			Interior Bearing Length	
	(psf)	(psf)	(in.^{4}/ft)	(in.^{4}/ft)	(in.^{3}/ft)	(in.^{3}/ft)	2"	3"	4"	4"	8"	2"	3"	4"	4"	8"
22	2.0	1.9	0.721	0.785	0.353	0.405	618	711	789	1240	1447	579	648	706	1448	1708
20	2.4	2.3	0.889	0.953	0.452	0.509	870	997	1105	1738	2154	871	971	1056	2066	2597
18	3.1	3.1	1.229	1.273	0.671	0.722	1481	1687	1860	2941	3682	1624	1797	1943	3574	4548
16	3.9	3.8	1.571	1.587	0.883	0.932	2240	2538	2789	4430	5497	2611	2873	3094	5458	6887

Notes:

1. Section properties are based on F_y = 50,000 psi.
2. I_d is for deflection due to uniform loads.
3. S_{eff} (+ or -) is the effective section modulus.
4. Multiply tabulated deck values listed above by the following adjustment factors to obtain acoustical deck section properties:

Deck Type	I_d for Deflection		Moment		Allowable Reactions per ft of Width (lb) for One Flange Loading (lb)	
	Single Span	Multi Span	$+S_{eff}$	$-S_{eff}$	End Bearing	Interior Bearing
N3 - Acoustical	0.93	0.94	0.91	0.92	1.00	0.85

5. Allowable (ASD) reactions are based on web crippling, per AISI S100 Section C3.4, where Ω_w = 1.70 for end bearing and 1.75 for interior bearing. Nominal reactions may be determined by multiplying the table values by Ω_w. LRFD reactions may be determined by multiplying nominal reactions by Φ_w = 0.90 for end reactions and 0.85 for interior reactions.

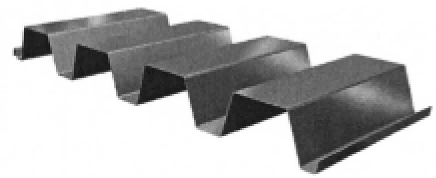
Type PLN3™ or HSN3™

Allowable Uniform Loads (psf)

SPAN	DECK GAGE	CRITERIA	SPAN (ft-in.) 4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"	16'-0"	17'-0"	18'-0"	19'-0"	20'-0"
SINGLE	22	Stress	300	282	196	144	110	87	71	58	49	42	36	31	28	24	22	20	18
		L/360	♦♦♦	252	146	92	62	43	32	24	18	14	12	9	8	6	5	5	4
		L/240	♦♦♦	♦♦♦	♦♦♦	138	92	65	47	36	27	22	17	14	12	10	8	7	6
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	87	63	47	37	29	23	19	15	13	11	9	8
	20	Stress	300	300	251	184	141	112	90	75	63	53	46	40	35	31	28	25	23
		L/360	♦♦♦	♦♦♦	180	113	76	53	39	29	23	18	14	12	10	8	7	6	5
		L/240	♦♦♦	♦♦♦	♦♦♦	170	114	80	58	44	34	27	21	17	14	12	10	9	7
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	107	78	58	45	35	28	23	19	16	13	11	10
	18	Stress	300	300	300	274	210	166	134	111	93	79	68	60	52	46	41	37	34
		L/360	♦♦♦	♦♦♦	249	157	105	74	54	40	31	24	20	16	13	11	9	8	7
		L/240	♦♦♦	♦♦♦	♦♦♦	235	158	111	81	61	47	37	29	24	20	16	14	12	10
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	148	108	81	62	49	39	32	26	22	18	16	13
	16	Stress	300	300	300	300	276	218	177	146	123	104	90	78	69	61	55	49	44
		L/360	♦♦♦	♦♦♦	♦♦♦	200	134	94	69	52	40	31	25	20	17	14	12	10	9
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	201	141	103	78	60	47	38	31	25	21	18	15	13
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	269	189	138	103	80	63	50	41	34	28	24	20	17
DOUBLE	22	Stress	300	300	225	165	127	100	81	67	56	48	41	36	32	28	25	22	20
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	62	48	38	30	25	20	17	14	12	10
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	30	25	21	18	16
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦
	20	Stress	300	300	283	208	159	126	102	84	71	60	52	45	40	35	31	28	25
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	100	75	58	46	37	30	25	20	17	15	13
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	45	37	31	26	22	19
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	25
	18	Stress	300	300	300	295	226	178	144	119	100	85	74	64	56	50	45	40	36
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	134	101	78	61	49	40	33	27	23	20	17
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	73	60	49	41	35	29	25
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	39	34
	16	Stress	300	300	300	300	291	230	186	154	129	110	95	83	73	64	58	52	47
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	229	167	126	97	76	61	50	41	34	29	24	21
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	91	74	61	51	43	37	31
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	57	49	42
TRIPLE	22	Stress	300	300	281	207	158	125	101	84	70	60	52	45	40	35	31	28	25
		L/360	♦♦♦	♦♦♦	♦♦♦	189	126	89	65	49	37	29	24	19	16	13	11	9	8
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	97	73	56	44	35	29	24	20	17	14	12
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	59	47	38	32	26	22	19	16
	20	Stress	300	300	300	260	199	157	127	105	88	75	65	57	50	44	39	35	32
		L/360	♦♦♦	♦♦♦	♦♦♦	229	154	108	79	59	45	36	29	23	19	16	13	11	10
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	118	89	68	54	43	35	29	24	20	17	15
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	72	57	47	38	32	27	23	20
	18	Stress	300	300	300	300	282	223	181	149	125	107	92	80	71	62	56	50	45
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	205	144	105	79	61	48	38	31	26	21	18	15	13
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	216	157	118	91	72	57	47	38	32	27	23	20
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	122	96	77	62	51	43	36	31	26
	16	Stress	300	300	300	300	300	288	233	193	162	138	119	104	91	81	72	65	58
		L/360	♦♦♦	♦♦♦	♦♦♦	♦♦♦	256	180	131	98	76	60	48	39	32	27	22	19	16
		L/240	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	269	196	148	114	89	72	58	48	40	34	29	25
		L/180	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	♦♦♦	151	119	95	78	64	53	45	38	33

See footnotes on page 81.

Type PLN3™ or HSN3™



Footnotes for Diaphragm Shear Strength and Flexibility Factor Tables

General Notes

1. VSC2 = Verco Sidelap Connection 2; BP = Button Punch; TSW = Top Seam Weld; #10 = #10 Generic Screw. Sidelap connections are not required at support locations.
2. The dimension from the first and last sidelap connection within each span is to be no more than one-half of specified spacing.
3. R is the ratio of vertical span (L_V) of the deck to the length (L_S) of the deck sheet: $R = L_V / L_S$.
4. Interpolation of diaphragm shear strength between adjacent spans or sidelap spacings is permissible. For interpolation of the diaphragm flexibility factor between adjacent spans, use the flexiblity factor for the closest adjacent span length.
5. Diaphragm shear values for side seam fasteners placed at spacings other than those in the table should be determined based on the number of fasteners in each span.
6. For web perforated acoustical deck profiles, modify tabulated q and F values using the following adjustment factors:

Deck Type	R_q	R_F
N3 - Acoustical	0.93	1.07

Note: Adjustment Factor, R_q must be applied only to allowable diaphragm shear strengths governed by panel buckling which are shown in the shaded areas of the diaphragm tables.

Notes Specific to Tables using Welds to Supports

1. The allowable diaphragm shear values in the table utilize a factor of safety, $\Omega = 3.0$ (limited by connections) with the exception of the gray shaded table values, which utilize a factor of safety of $\Omega = 2.0$ (limited by panel buckling).
2. A 1" x 3/8" effective arc seam weld is required at supports adjacent to sidelap and a 1/2" effective diameter arc spot welds are required at supports in interior flutes.

Notes Specific to Tables using Hilti or Pneutek Fasteners to Supports

1. X-EDNK22 = Hilti EDNK22 THQ12 fastener; X-ENP-19 = Hilti X-ENP-19 L15 fastener; K66 = Pneutek K66062 or K66075 fasteners; K64 = Pneutek K64062 fastener; SDK63 = Pneutek SDK63075; SDK61 = Pneutek SDK61075
2. The allowable diaphragm shear values in the table utilize a factor of safety, $\Omega = 2.5$ (limited by connections) with the exception of the shaded table values, which utilize a factor of safety of $\Omega = 2.0$ (limited by panel buckling).

Notes Specific to Tables using Screws to Supports

1. The allowable diaphragm shear values in the table utilize a factor of safety, $\Omega = 2.5$ (limited by connections) with the exception of the shaded table values, which utilize a factor of safety of $\Omega = 2.0$ (limited by panel buckling).
2. Deck is attached with minimum #12 Screws (self drilling, self tapping) to supports. Select appropriate screw based on actual substrate thickness. This table is provided as a guide, proper selection should be verified based on the specific fasteners used.

Support Thickness	Fastener Designation
33 mil (0.0346") to 3/16"	#3 Drill Point
1/8" to 1/4"	#4 Drill Point
1/8" to 1/2"	#5 Drill Point

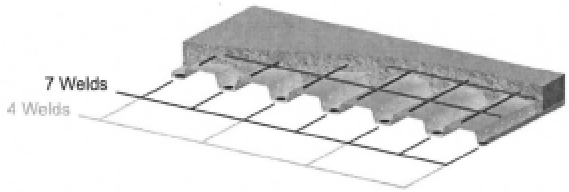
3. All tabulated diaphragm values shown in this section are for a minimum 0.0385 in. thick support with SDI recognized screws produced by Buildex, Elco, Hilti or Simpson Strong-Tie. If the minimum support thickness can not be met or a screw that is not recognized by SDI is used, modify tabulated q and F values based on actual substrate and thickness using Adjustment Factors listed in this table.

Deck Gage	Factors	Substrate Thickness and Strength									
		20 ga 33 mil (0.0345 in)		18 ga 43 mil (0.0451 in)		16 ga 54 mil (0.0566 in)		14 ga 68 mil (0.0713 in)		≥ 12 ga ≥ 97 mil (0.1017 in)	
		33 ksi	50 ksi	33 ksi	50 ksi	33 ksi	50 ksi	33 ksi	50 ksi	33 ksi	50 ksi
22	R_q	0.44	0.61	0.67	0.78	0.78	0.78	0.78	0.78	0.78	0.78
	R_F	1.28	1.25	1.17	1.00	1.00	1.00	1.00	1.00	1.00	1.00
20	R_q	0.34	0.49	0.54	0.74	0.74	0.78	0.78	0.78	0.78	0.78
	R_F	1.31	1.31	1.24	1.19	1.15	1.00	1.00	1.00	1.00	1.00
18	R_q	0.26	0.37	0.38	0.55	0.55	0.78	0.76	0.78	0.78	0.78
	R_F	1.34	1.39	1.30	1.31	1.26	1.18	1.19	1.00	1.00	1.00
16	R_q	0.20	0.30	0.30	0.44	0.43	0.65	0.61	0.78	0.78	0.78
	R_F	1.43	1.66	1.39	1.54	1.33	1.34	1.25	1.00	1.00	1.00

4. Adjustment factors are based on connection strengths determined in accordance with Section E4 of AISI S100.
These self drilling, self tapping screws must be compliant with ASTM C1315.
5. Allowable Diaphragm Strength = $q \cdot R_q$; Flexibility Factor = $F \cdot R_F$.
6. These adjustment factors are based on the maximum adjustment for the tabulated span lengths and fastener patterns. To calculate a specific condition, use design equations listed at the end of Evaluation Report ER-0217.

PLB™ or B FORMLOK™

- **3½ in. TOTAL SLAB DEPTH**
- **Normal Weight Concrete (145 pcf)**
  30.6 psf
- **Galvanized or Phosphatized/Painted**



B 3½" NW

Deck Weight and Section Properties

Gage	Weight (psf) Galv G60	Weight (psf) Phos/ Painted	I_d for Deflection Single Span (in.⁴/ft)	I_d for Deflection Multiple Spans (in.⁴/ft)	Moment $+S_{eff}$ (in.³/ft)	Moment $-S_{eff}$ (in.³/ft)	Allowable Reactions per ft of Width (lb) End Bearing 2"	End Bearing 3"	End Bearing 4"	Interior Bearing 3"	Interior Bearing 4"
22	1.9	1.8	0.177	0.192	0.176	0.188	935	1076	1163	1559	1671
20	2.3	2.2	0.219	0.231	0.230	0.237	1301	1492	1609	2190	2340
18	2.9	2.8	0.302	0.306	0.314	0.331	2181	2484	2667	3714	3950
16	3.5	3.4	0.381	0.381	0.399	0.410	3265	3699	3955	5607	5938

Allowable Superimposed Loads (psf)

Gage	Spans	Max. UCS[1]	Span (ft-in.) 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"
22	1	6'-6"	261	228	170	148	130	115	101	90	80	71	64
	2	7'-8"	261	228	202	180	130	115	101	90	80	71	64
	3	7'-9"	261	228	202	180	130	115	101	90	80	71	64
20	1	7'-9"	274	240	212	189	138	122	108	96	85	76	68
	2	9'-1"	274	240	212	189	170	153	140	96	85	76	68
	3	9'-3"	274	240	212	189	170	153	140	96	85	76	68
18	1	8'-10"	297	260	230	205	184	166	119	106	95	85	76
	2	10'-8"	297	260	230	205	184	166	151	138	127	117	76
	3	11'-0"	297	260	230	205	184	166	151	138	127	117	108
16	1	9'-6"	297	260	230	205	184	166	151	138	94	84	75
	2	11'-10"	297	260	230	205	184	166	151	138	127	117	108
	3	11'-7"	297	260	230	205	184	166	151	138	127	117	108

[1] Max. UCS = Maximum Unshored Clear Span (ft-in.)

Shoring required in shaded areas to right of heavy line.

Allowable Diaphragm Shear Values, q (plf) and Flexibility Factors, F (in./lb x 10⁶)

Gage	Welds	Span (ft-in.) 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"
22	q4	1825	1787	1754	1726	1701	1679	1659	1642	1626	1612	1599
	F4	0.45	0.46	0.47	0.48	0.48	0.49	0.50	0.50	0.51	0.51	0.52
	q7	2035	1981	1934	1893	1858	1827	1799	1774	1752	1732	1713
	F7	0.41	0.42	0.43	0.44	0.44	0.45	0.46	0.46	0.47	0.48	0.48
20	q4	1893	1847	1808	1773	1743	1717	1694	1673	1654	1637	1621
	F4	0.40	0.41	0.42	0.42	0.43	0.44	0.44	0.45	0.45	0.46	0.46
	q7	2145	2079	2023	1975	1932	1895	1861	1832	1805	1780	1758
	F7	0.35	0.36	0.37	0.38	0.39	0.40	0.40	0.41	0.42	0.42	0.43
18	q4	2046	1985	1932	1887	1847	1812	1781	1753	1728	1705	1684
	F4	0.32	0.33	0.34	0.35	0.35	0.36	0.37	0.37	0.38	0.38	0.39
	q7	2381	2294	2219	2155	2098	2048	2004	1964	1929	1896	1867
	F7	0.27	0.28	0.29	0.30	0.31	0.32	0.33	0.33	0.34	0.34	0.35
16	q4	2215	2138	2073	2016	1966	1922	1883	1848	1816	1788	1762
	F4	0.26	0.27	0.28	0.29	0.30	0.30	0.31	0.32	0.32	0.33	0.33
	q7	2634	2525	2432	2351	2280	2218	2162	2113	2068	2027	1991
	F7	0.22	0.23	0.24	0.25	0.26	0.26	0.27	0.28	0.28	0.29	0.29

Joist LRFD Load Tables – DLH-Series

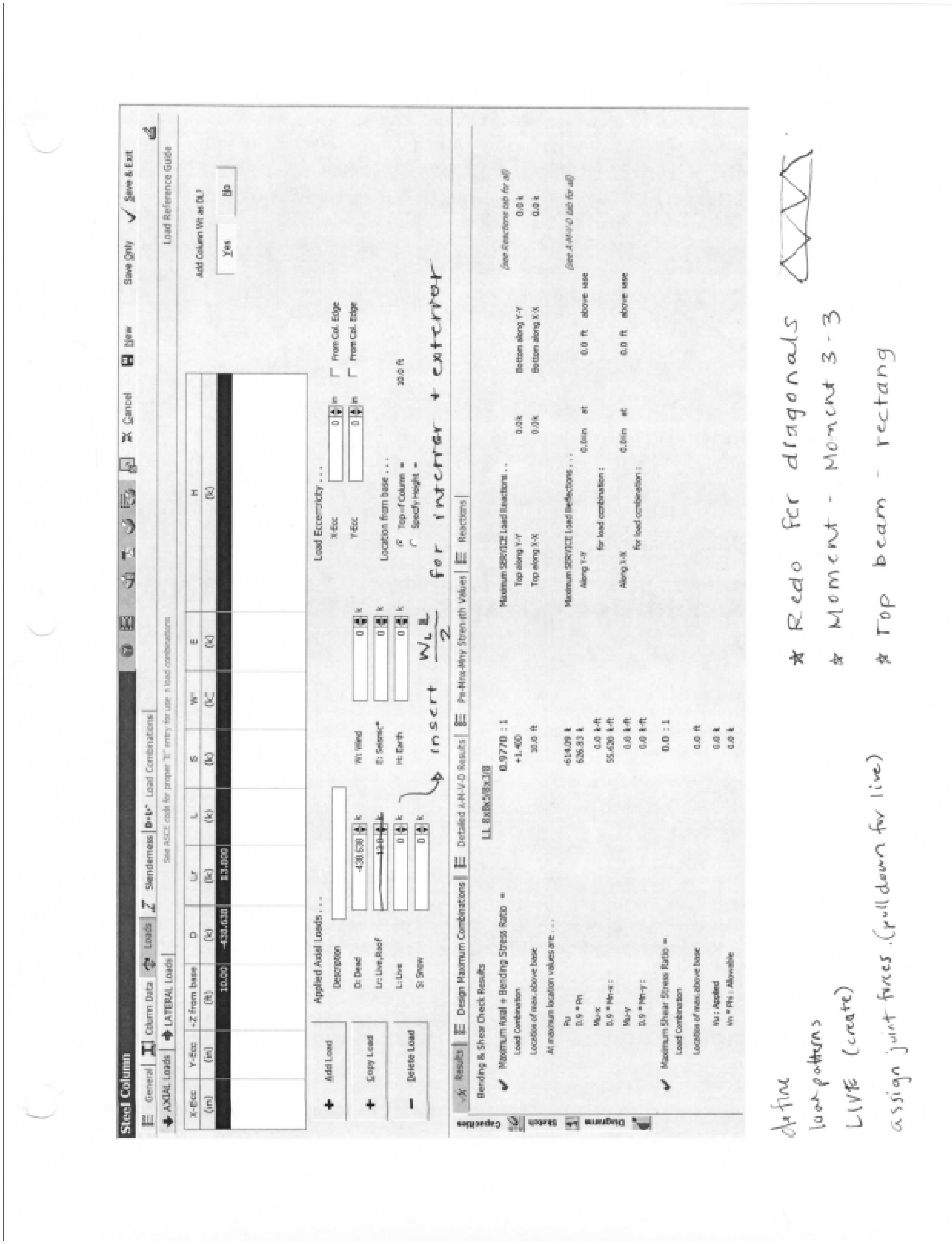
LRFD

STANDARD LOAD TABLE FOR OPEN WEB STEEL JOISTS, DLH-SERIES
BASED ON 50 KSI YIELD

JOIST DESIGNATION	APPROX. WT. (lbs/ft)	DEPTH (in)	MAX. LOAD (plf)	SAFE LOAD* (pounds)		LOADS SHOWN IN POUNDS PER LINEAR FOOT (plf)														
		SPAN IN FEET	< 62	62-89		90	91	92	93	94	95	96	97	98	99	100	101	102	103	104
52DLH10	25	52	648	40,200		447 171	436 165	427 159	418 154	409 150	400 145	391 140	384 136	376 132	369 128	361 124	354 120	346 116	340 114	334 110
52DLH11	26	52	712	44,130		490 187	480 181	469 174	459 169	448 164	439 158	430 153	421 149	412 144	405 140	396 135	388 132	381 128	373 124	366 120
52DLH12	29	52	794	49,230		547 204	536 197	523 191	513 185	501 179	490 173	480 168	471 163	460 158	451 153	442 149	433 144	426 140	417 135	409 132
52DLH13	34	52	964	59,760		664 247	649 239	636 231	621 224	609 216	595 209	583 203	571 197	559 191	549 185	537 180	526 174	516 170	507 164	496 159
52DLH14	39	52	1,103	68,370		760 276	745 266	729 258	714 249	699 242	685 234	670 227	657 220	645 213	631 207	619 201	607 194	595 189	585 184	573 178
52DLH15	42	52	1,239	76,800		853 311	835 301	817 291	799 282	783 272	766 264	750 256	735 247	720 240	705 233	691 226	676 219	664 213	651 207	639 201
52DLH16	45	52	1,335	82,800		921 346	901 335	882 324	862 314	844 304	826 294	810 285	792 276	777 267	760 260	745 252	730 245	717 237	702 230	688 224
52DLH17	52	52	1,537	95,310		1,059 335	1,036 381	1,014 369	991 357	970 346	951 335	930 324	912 315	892 304	874 296	856 286	840 279	823 270	808 263	792 255
		SPAN IN FEET	<67	67-97		98	99	100	101	102	103	104	105	106	107	108	109	110	111	112
56DLH11	26	56	631	42,300		432 169	424 163	415 158	408 153	400 149	393 145	385 140	379 136	372 133	366 129	358 125	352 122	346 118	340 115	334 113
56DLH12	30	56	725	48,600		496 184	486 178	477 173	468 168	459 163	450 158	442 153	433 150	426 145	417 141	409 137	402 133	394 130	388 126	381 123
56DLH13	34	56	879	58,600		601 223	591 216	579 209	568 204	558 197	547 191	537 186	526 181	516 175	507 171	496 166	487 161	478 157	471 152	462 149
56DLH14	39	56	993	66,540		679 249	666 242	652 234	640 228	628 221	616 214	604 209	594 202	582 196	571 190	562 186	552 181	541 175	532 171	523 167
56DLH15	42	56	1,135	76,020		777 281	762 272	747 264	732 256	717 248	703 242	690 234	676 226	664 221	651 215	639 209	628 204	616 198	604 192	594 188
56DLH16	46	56	1,224	82,020		836 313	822 304	805 294	789 285	774 277	759 269	744 262	730 254	717 247	703 240	690 233	678 227	666 221	654 214	642 209
56DLH17	51	56	1,411	94,530		964 356	945 345	927 335	907 325	891 316	873 308	856 298	840 289	823 281	808 273	793 266	780 258	765 251	751 245	738 238
		SPAN IN FEET	< 71	71-99	100-105	106	107	108	109	110	111	112	113	114	115	116	117	118	119	120
60DLH12	29	60	659	46,800	46,800	442 168	433 163	426 158	418 154	411 150	405 146	397 142	391 138	384 134	378 131	372 128	366 124	360 121	354 118	348 115
60DLH13	35	60	801	56,880	56,880	537 203	526 197	517 191	508 187	499 181	490 176	483 171	474 167	466 163	459 158	451 154	444 151	436 147	429 143	423 139
60DLH14	40	60	890	63,210	63,210	597 216	586 210	574 205	564 199	555 193	544 189	534 183	525 178	516 173	507 170	498 165	490 161	481 156	474 152	465 149
60DLH15	43	60	1,045	74,190	74,190	700 256	687 248	675 242	663 235	651 228	640 223	628 216	618 210	607 205	597 200	588 194	577 190	568 185	559 180	550 175
60DLH16	46	60	1,149	81,570	81,570	769 285	756 277	741 269	727 262	714 255	702 247	690 241	676 235	666 228	654 223	642 217	631 211	621 206	610 201	600 196
60DLH17	52	60	1,320	93,750	93,750	886 324	868 315	853 306	837 298	822 290	807 283	793 275	778 267	765 261	751 254	739 247	726 241	714 235	702 228	690 223
60DLH18	59	60	1,524	108,180	108,180	1,021 366	1,002 357	984 346	966 337	948 327	931 319	915 310	898 303	883 294	867 286	852 279	838 272	823 266	810 259	796 252
		SPAN IN FEET	<76	76-99	100-113	114	115	116	117	118	119	120	121	122	123	124	125	126	127	128
64DLH12	31	64	594	45,120	45,120	396 153	388 150	382 146	376 142	370 138	364 135	358 132	352 129	346 126	342 122	336 119	331 116	327 114	321 111	316 109
64DLH13	34	64	720	54,750	54,750	481 186	472 181	465 176	457 171	450 168	442 163	436 159	429 155	421 152	415 148	409 144	403 141	396 137	390 134	385 131
64DLH14	40	64	825	62,730	62,730	550 199	540 193	531 189	523 184	514 179	505 174	498 171	489 166	481 162	474 158	466 154	459 151	451 147	444 143	438 140
64DLH15	43	64	946	71,910	71,910	631 234	621 228	610 223	600 217	591 211	580 206	571 201	562 196	553 191	544 187	537 182	528 177	520 173	511 170	504 165
64DLH16	46	64	1,065	80,940	80,940	711 262	699 254	687 248	675 242	664 235	652 229	642 224	631 219	621 213	610 208	601 203	591 198	582 193	573 189	564 184
64DLH17	52	64	1,227	93,270	93,270	819 298	804 290	790 283	777 275	763 268	751 262	738 255	726 248	714 243	702 237	691 231	681 226	669 220	658 215	648 210
64DLH18	59	64	1,417	107,700	107,700	945 337	928 328	912 320	897 311	880 304	867 298	852 289	838 282	823 274	810 267	798 261	784 255	772 249	760 243	748 237

Appendix F

SAP 2000 and ENERCALC iterations used to determine steel truss member sizes



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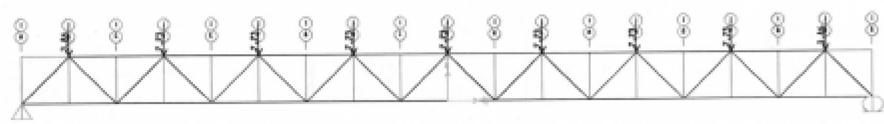
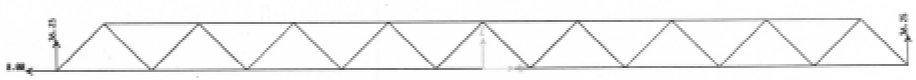
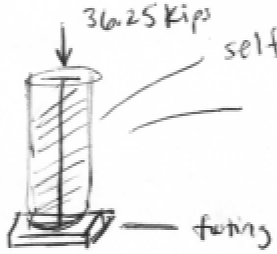
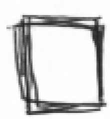


SAP2000 v15.1.0 - File:Truss_Design5 - Joint Loads (DEAD) (As Defined) - Kip, ft, F Units



column design





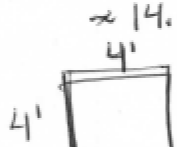
$$\frac{36.25\text{kips}}{1} \cdot \frac{1000\text{lbs}}{1\text{kip}} \cdot \frac{\text{ft}^2}{2500\text{lb}}$$

→ bearing pressure allowable

DL = 2000 psf

DL+LL = 2500psf

$\approx 14.5\text{ft}^2$ roundup to 16ft^2



4x4 square footing

SAP2000 v15.1.0 - File:Truss_Design5 - Joint Reactions (DEAD) - Kip, ft, F Units

Steel Column

File = \\samba1\anon\docengr\Desktop\SGP4RB~PITX1GK3~K.EC6
ENERCALC, INC. 1983-2014, Build:6.14.6.15, Ver:6.14.6.15

Lic. # : KW-06090157 - Educational Version — Licensed User : SANTA CLARA UNIVERSITY, CIVIL ENGINEERING

Description : --None--

Code References

Calculations per AISC 360-10, IBC 2012, CBC 2013, ASCE 7-10
Load Combinations Used : ASCE 7-05

General Information

Steel Section Name : LL 8x8x5/8x3/8
Analysis Method : Load Resistance Factor
Steel Stress Grade
Fy : Steel Yield 36.0 ksi
E : Elastic Bending Modulus 29,000.0 ksi
Load Combination : ASCE 7-05

Overall Column Height 10.0 ft
Top & Bottom Fixity Top & Bottom Pinned
Brace condition for deflection (buckling) along columns :
X-X (width) axis :
Fully braced against buckling along X-X Axis
Y-Y (depth) axis :
Fully braced against buckling along Y-Y Axis

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 660.0 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, D = -438.638, LR = 13.80 k
BENDING LOADS . . .
Moment acting about X-X axis, D = 8.053 k-ft
Moment acting about X-X axis, D = -8.053 k-ft

DESIGN SUMMARY

Bending & Shear Check Results

PASS Max. Axial+Bending Stress Ratio = 0.9770 : 1
Load Combination +1.40D
Location of max.above base 10.0 ft
At maximum location values are . . .
Pu -614.09 k
0.9 * Pn 626.83 k
Mu-x 0.0 k-ft
0.9 * Mn-x : 55.620 k-ft
Mu-y 0.0 k-ft
0.9 * Mn-y : 0.0 k-ft

PASS Maximum Shear Stress Ratio = 0.0 : 1
Load Combination
Location of max.above base 0.0 ft
At maximum location values are . . .
Vu : Applied 0.0 k
Vn * Phi : Allowable 0.0 k

Maximum SERVICE Load Reactions . .
Top along X-X 0.0 k
Bottom along X-X 0.0 k
Top along Y-Y 0.0 k
Bottom along Y-Y 0.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y 0.0 in at 0.0ft above base
for load combination :
Along X-X 0.0 in at 0.0ft above base
for load combination :

Load Combination Results

Load Combination	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+1.40D	0.977	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+0.50Lr+1.60L+1.60H	0.826	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60L+0.50S+1.60H	0.837	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60Lr+0.50L	0.802	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60Lr+0.80W	0.802	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+0.50L+1.60S	0.837	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+1.60S+0.80W	0.837	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+0.50Lr+0.50L+1.60W	0.826	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+0.50L+0.50S+1.60W	0.837	PASS	10.00 ft	0.000	PASS	0.00 ft
+1.20D+0.50L+0.20S+E	0.837	PASS	10.00 ft	0.000	PASS	0.00 ft
+0.90D+1.60W+1.60H	0.628	PASS	10.00 ft	0.000	PASS	0.00 ft
+0.90D+E+1.60H	0.628	PASS	10.00 ft	0.000	PASS	0.00 ft

Steel Column

File = \\sambe1\anon\dcengr\Desktop\SGP4RB~P\TX1GK3~K.EC6
ENERCALC, INC. 1983-2014, Build:6.14.6.15, Ver:6.14.6.15

Lic. # : KW-06090157 - Educational Version

Licensed User : SANTA CLARA UNIVERSITY, CIVIL ENGINEERING

Description : --None--

Maximum Reactions - Unfactored

Note: Only non-zero reactions are listed.

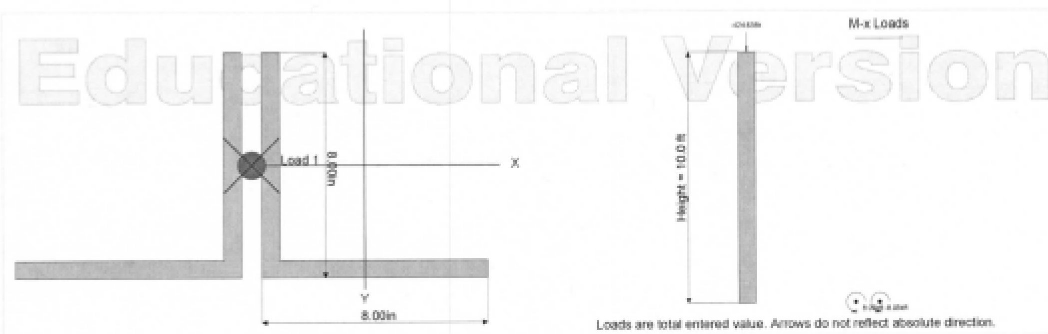
Load Combination	X-X Axis Reaction @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base
D Only		k		k	437.978 k
Lr Only		k		k	13.800 k
D+Lr		k		k	424.178 k

Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
D+Lr	0.0000 in	0.000 ft	0.000 in	0.000 ft

Steel Section Properties : LL 8x8x5/8x3/8

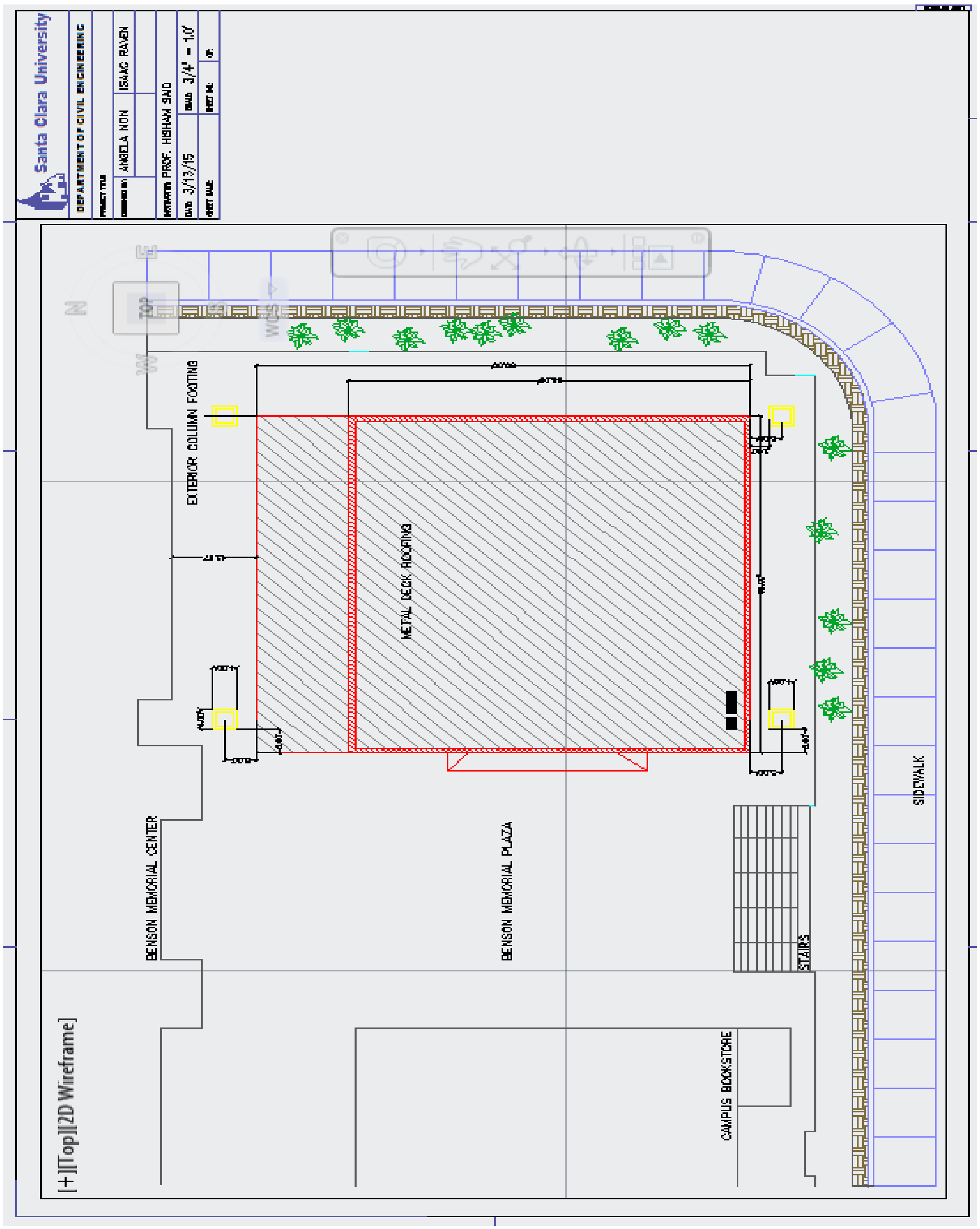
Depth	=	8.000 in	I xx	=	119.00 in^4	J	=	2.600 in^4
			S xx	=	20.60 in^3			
Leg Width	=	8.000 in	R xx	=	2.480 in			
Thickness	=	0.625 in	Zx	=	37.100 in^3	H	=	0.833 in
Area	=	19.400 in^2	I yy	=	230.909 in^4			
Weight	=	66.000 plf	S yy	=	28.203 in^3			
			R yy	=	3.450 in			
Ycg	=	5.790 in	Qs	=	0.997			
Xcg	=	8.188 in						
Leg Spacing	=	0.375 in						

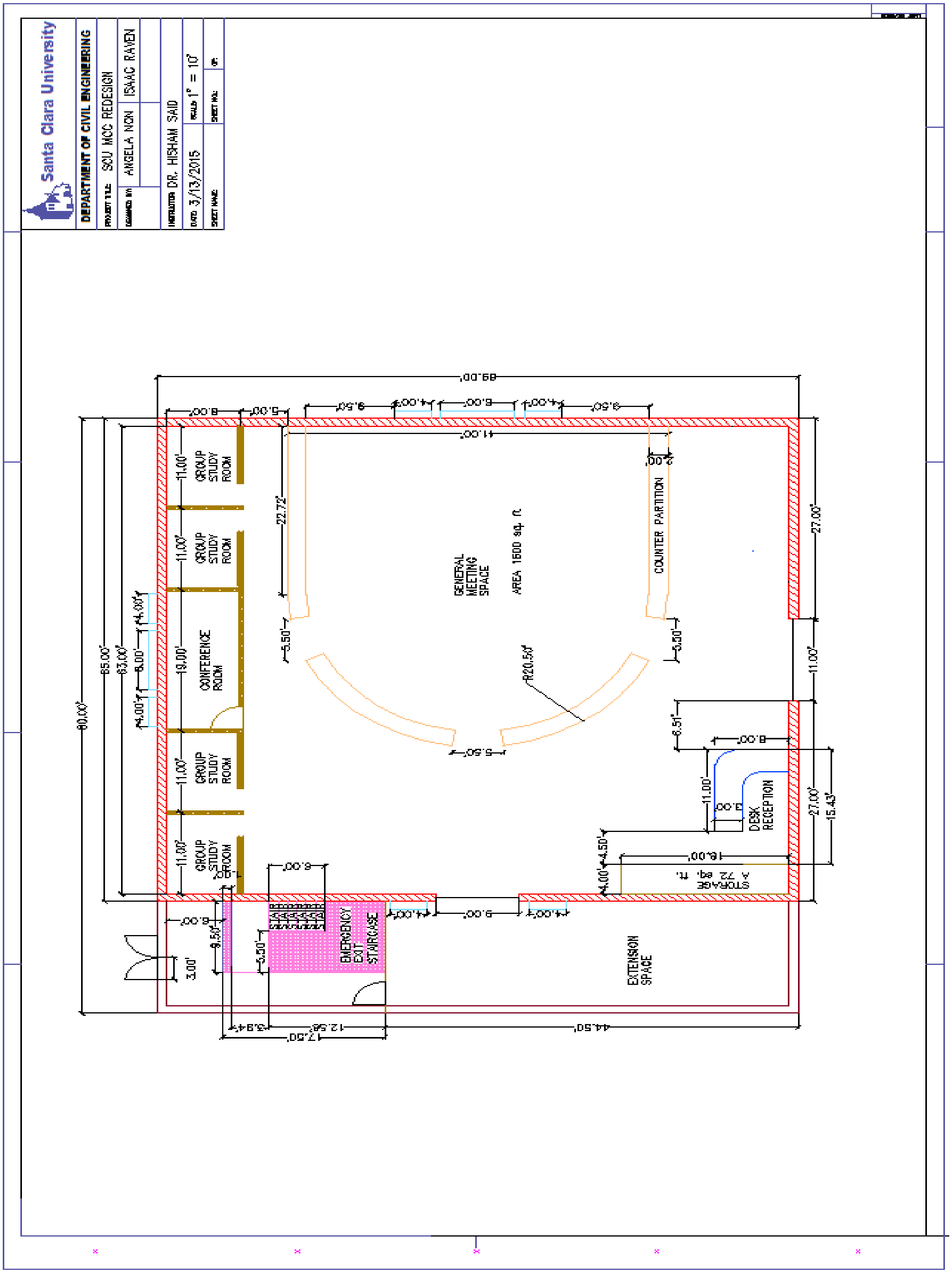


Loads are total entered value. Arrows do not reflect absolute direction.

Appendix G

Architectural drawings of the interior layout, as created through Autodesk AutoCAD 2014





Santa Clara University
DEPARTMENT OF CIVIL ENGINEERING
PROJECT TITLE: SCU MCC REDESIGN
DRAWN BY: ANGELA NON ISAAC RAVEN
INSTRUCTOR: DR. HISHAM SAID
DATE: 3/13/2015
SCALE: 1" = 10'
GROUP STUDY ROOM
CONFERENCE ROOM
GENERAL MEETING SPACE
AREA 1600 sq. ft.
COUNTER PARTITION
R20.50'
DESK RECEPTION
STORAGE A 72 sq. ft.
EMERGENCY EXIT STAIRCASE
STAIR
EXTENSION SPACE
69.00'
80.00'
65.00'
63.00'
44.50'
17.50'
12.58'
27.00'
15.43'
22.72'

Appendix H

Additional 3D models of the proposed redesign, as provided through Revit 2014.

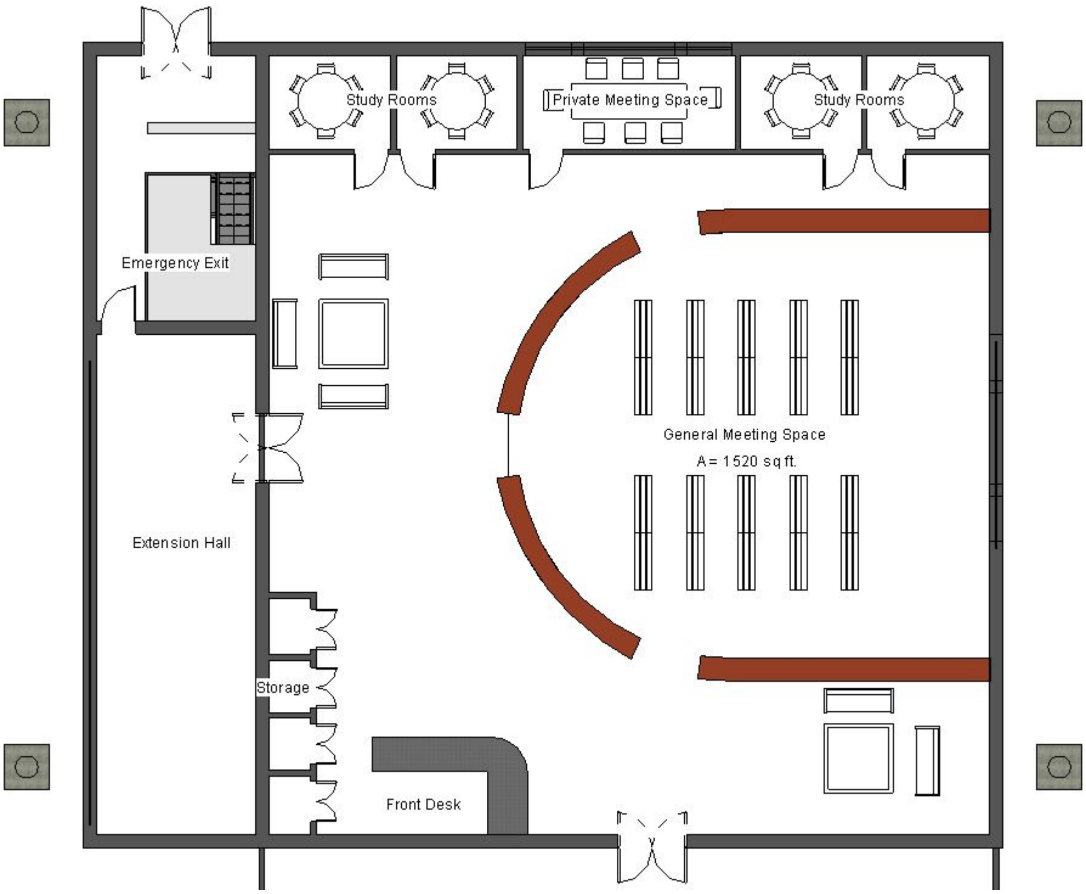


Figure G-1. Labeled plan view of extension and original structure

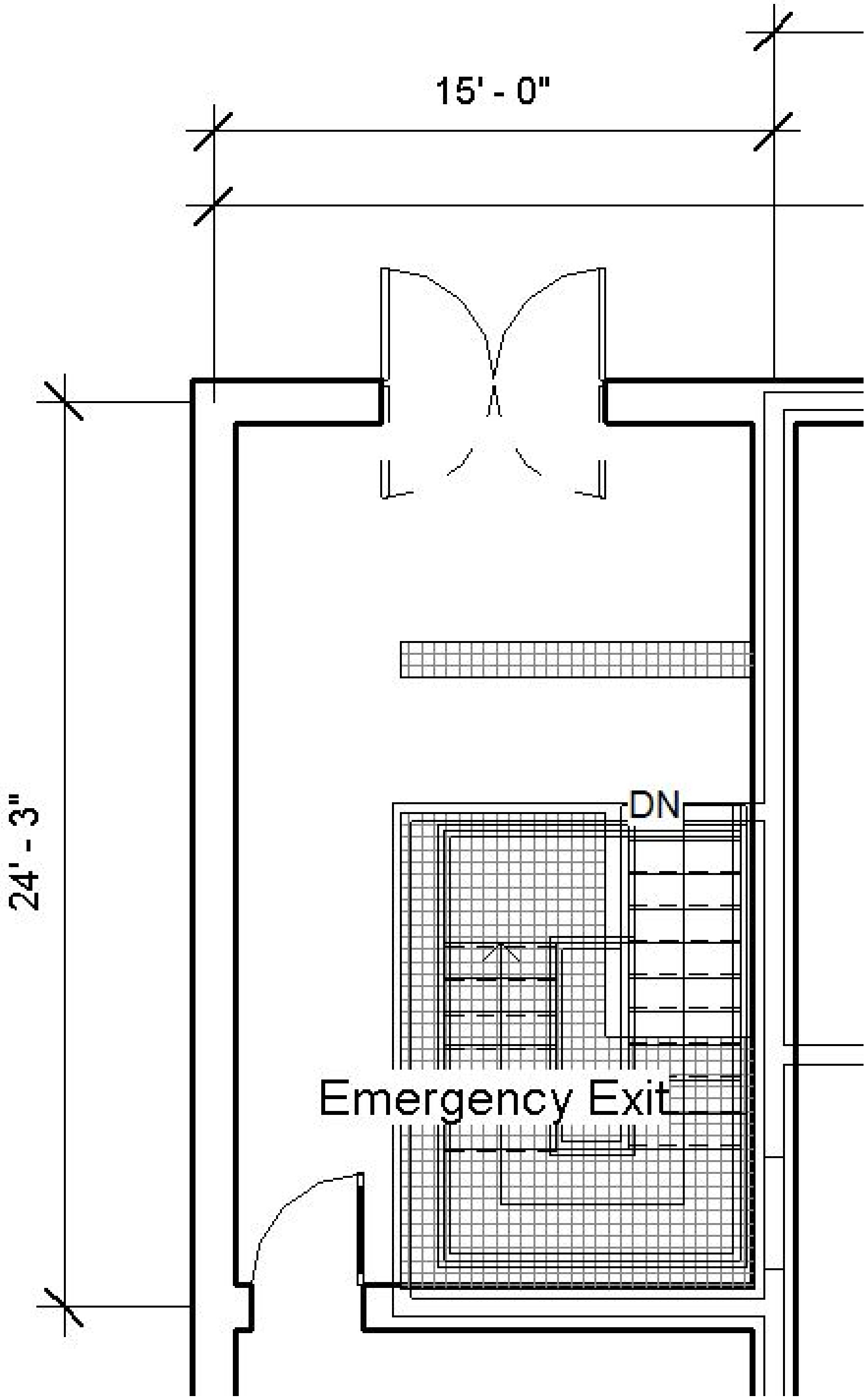


Figure G-2. Detailed emergency exit continuation plans

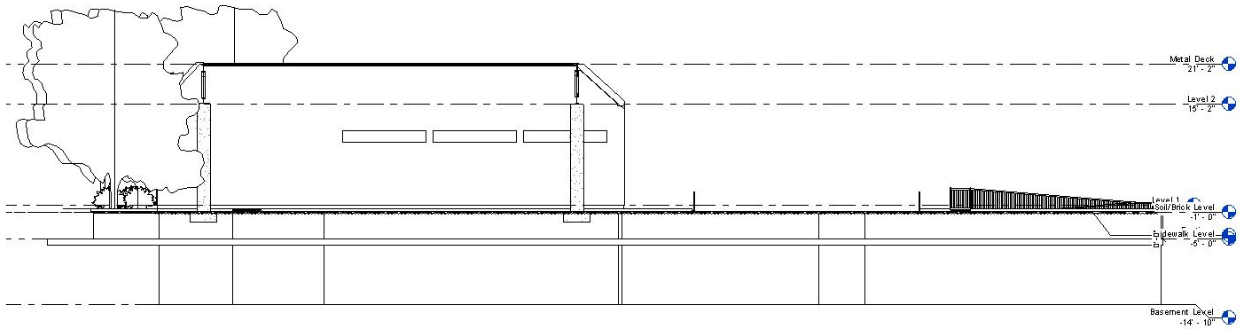


Figure G-3. Northeast elevation view of structure.

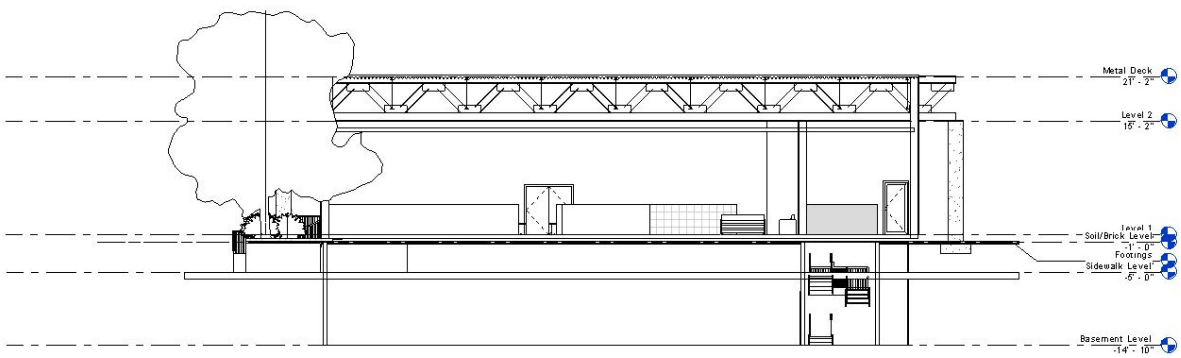


Figure G-4. Northwest elevation view

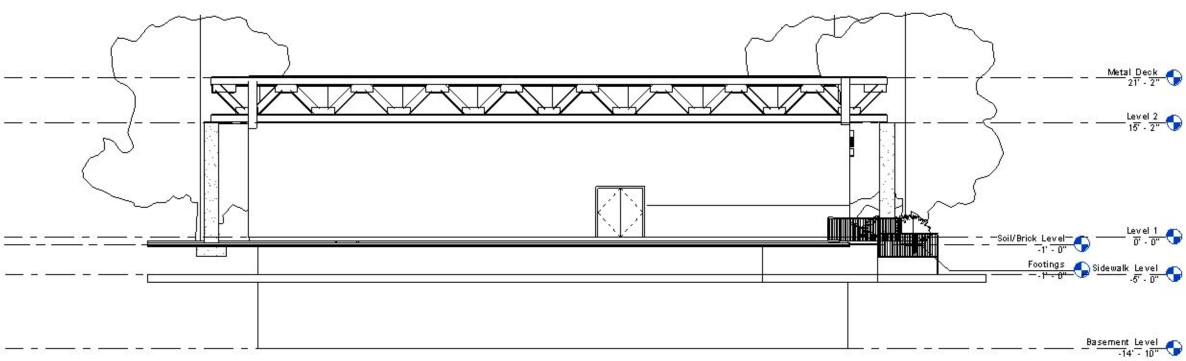


Figure G-5. Southeast elevation view

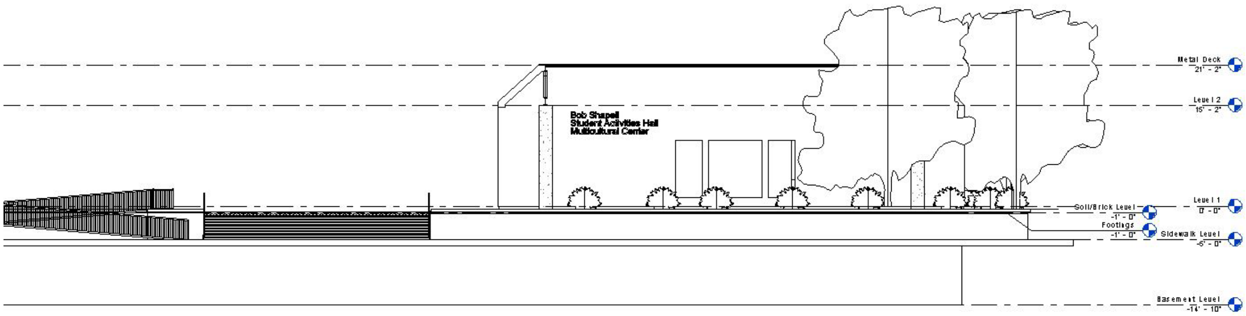


Figure G-6. Southwest elevation view

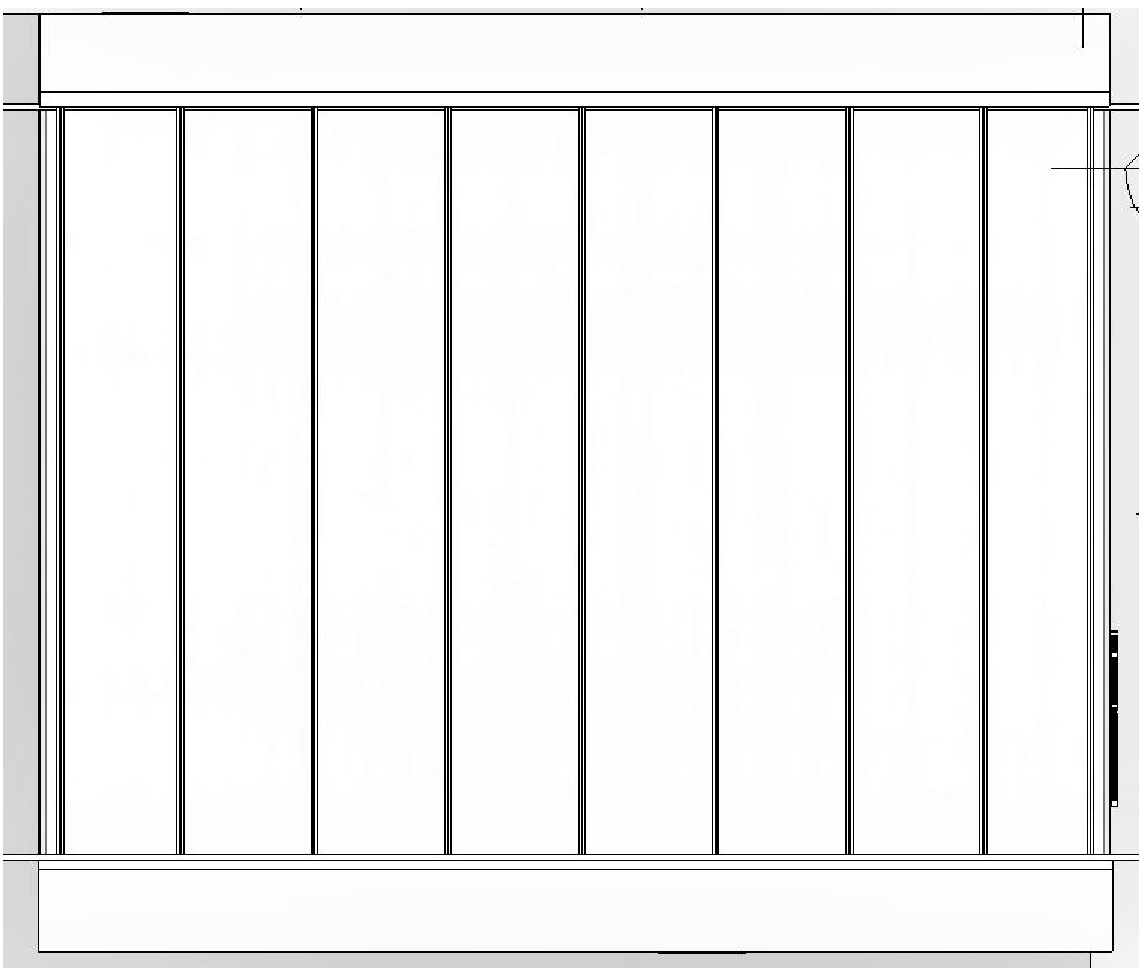


Figure 7. Truss joist roof plan

Appendix I

Additional work breakdown structures for demolition and architectural systems

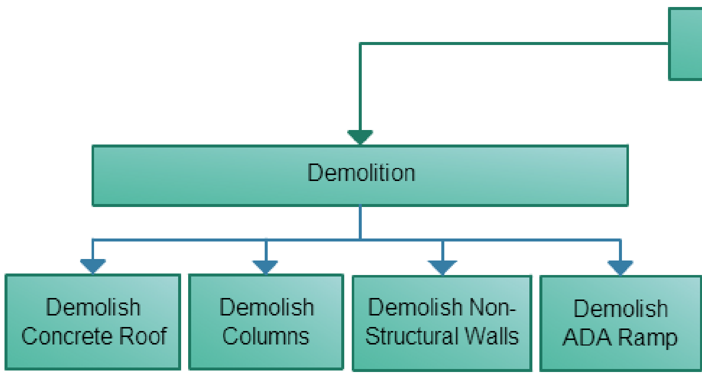


Figure I-1. Completed WBS for Demolition

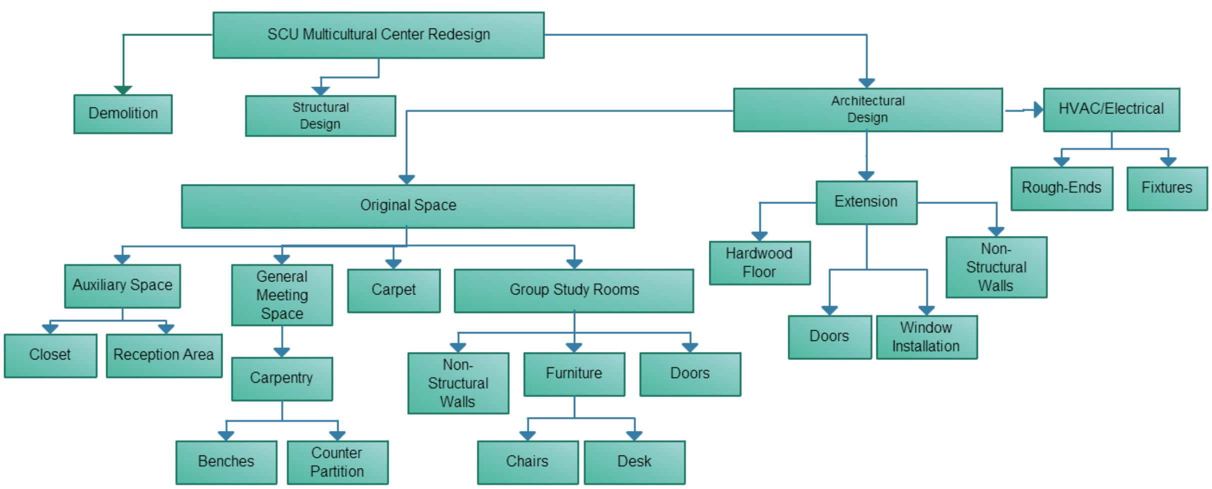


Figure I-2. Completed WBS for Architectural System

Appendix J

Complete Cost Estimate Breakdown

Cost Estimating Spreadsheet (RSMeans Building Construction Cost Data 2012)

ID	Activity	Quantity	unit	Daily Output	Duration days	Material $/Unit	WTC	Material Cost	Crew	Labor $/Unit	Labor FC	Labor Cost	Equip. $/Unit	Equip. C	Equip. Cost	Total Cost	Means Item	Page
1	Metal Decking (Roof)	5520	S.F.	4300	1.28	1.47	1.041	$ 8,445	E-4	0.37	1.400	$ 2,860	0.03	1.077	$ 178	$ 11,483	05 31 23.50 2700	139
2	Steel Truss Joists (installation and material)	621	L.F.	2000	0.31	41	1.093	$ 27,823	E-7	1.95	1.400	$ 1,695	0.87	1.077	$ 582	$ 30,100	05 21 13.50 3260	136
3	Demolish Non-Structural Walls (Drywall)	23	Ea.	24	0.96		1.477	$ -	1 Clab	11.7	1.303	$ 351		1.002	$ -	$ 351	02 41 19.16 6100	32
4	Steel Trusses Material	11.26	ton			2137	0.957	$ 23,028								$ 23,028		
5	Steel Strusses Installation				1.00			$ -	E-7		1.30000	$ 5,643			$ -	$ 5,643		
6	Demolish Structural Columns	240	C.F.	11300	0.02		1.273	$ -	B-3	0.16	1.21000	$ 46	0.2	1.21	$ 58	$ 105	02 41 16.13 0600	30
7	Demolish Roof	7475	C.F.	11300	0.66		0.000	$ -	B-3	0.16	1.21000	$ 1,447	0.2	1.21	$ 1,809	$ 3,256	02 41 16.13 0600	30
8	Mobilize - Crane, up to 75 ton	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 75	62	1.091	$ 68	$ 143	01 54 36.50 2000	21
9	Mobilize - Forklift	0.5	Ea.	7.2	0.07		0.000	$ -	1 Eqhv	53	1.41830	$ 38	62	1.091	$ 34	$ 71	02 54 36.50 2000	21
10	Mobilize - Hydraulic Pump	0.333333333	Ea.	7.2	0.05		0.000	$ -	1 Eqhv	53	1.41830	$ 25	62	1.091	$ 23	$ 48	02 54 36.50 2000	21
11	Demobilize - Crane, up to 75 ton	1	Ea.	7.2	0.14		0.000	$ -	1 Eqhv	53	1.41830	$ 75	62	1.091	$ 68	$ 143	01 54 36.50 2000	21
12	Demobilize - Forklift	0.5	Ea.	7.2	0.07		0.000	$ -	1 Eqhv	53	1.41830	$ 38	62	1.091	$ 34	$ 71	02 54 36.50 2000	21
13	Demobilize - Hydraulic Pump	0.333333333	Ea.	7.2	0.05		0.000	$ -	1 Eqhv	53	1.41830	$ 25	62	1.091	$ 23	$ 48	02 54 36.50 2000	21
14	Structural Wall Reshoring	4020	S.F.	1400	2.87	0.5	1.266	$ 2,544	2 Carp	0.4	1.71210	$ 2,753		1.317	$ -	$ 5,297	03 15 05.70 1500	62
15	Column Foundation Excavation	13	B.C.Y	150	0.09				B-11C	4.36	1.30260	$ 74	2.23	1.317	$ 34	$ 108	31 23 16.13 0050	577
16	Column Formwork	480	S.F.C.A	216	2.22	1.41	1.268	$ 858	C-1	7.05	1.92270	$ 6,506		1.091	$ -	$ 7,365	03 11 13.25 6500	54
17	Concrete Columns	7	C.Y.	51.85	0.14	257	1.354	$ 2,437	C-14A	171	1.73030	$ 2,071	14.35	1.331	$ 134	$ 4,642	03 30 53.40 1400	72
18	Column Foundation Formwork	216	S.F.C.A	160	1.35	2.93	1.268	$ 803	C-2	12.9	1.21000	$ 3,372		1.21	$ -	$ 4,174	03 11 13.35 0020	55
19	Column Footing Concrete	9	C.Y.			102	1.266	$ 1,162								$ 1,162	03 31 05.35 0150	74
20	Placing Footing Concrete	9	C.Y.	160	0.06				C-20	15.1	1.71210	$ 233	4.82	1.317	$ 57	$ 290	03 31 05.70 2150	75
21	Interior Walls	1620	S.F.	630	2.57	3.41	1	$ 5,524	J-4	11.8	2.01370	$ 38,494	0.42	1.002	$ 682	$ 44,700	09 21 13.10 0600	302
22	Expansion Wood Flooring	1035	S.F.	400	2.59	0.86	1.13580675	$ 1,011	1-Clab	0.7	1.6484	$ 1,194				$ 2,205	09 64 29.10 7500	321
23	Interior Carpeting	500	S.Y.	300	1.67	20	1.1688885	$ 11,689	4-Tilf	17.4	1.6484	$ 14,341				$ 26,030	09 68 16.10 3680	328
24	Conference Room Door	2	Ea.	14	0.14	490	1.1581875	$ 1,135	2-Carp	50.5	1.9955	$ 202				$ 1,337	08 14 13.10 3100	251
25	Conference Room Door Hardware	1	Ea.	1	1.00	94	1.12393125	$ 106	1-Carp	44	2.0137	$ 89				$ 194		
26	Expansion Exit Door	3	Ea.	15	0.20	645	1.0407375	$ 2,014	2-Carp	47	1.4001	$ 197				$ 2,211	08 13 13.25 2900	251
27	Expansion Exterior Wall	2175	S.F.	315	6.90	3.41	1.12393125	$ 8,336	J-2	5.9	2.0137	$ 25,841	0.42	1.549	$ 1,415	$ 35,592	09 21 13.10 0600	302
28	Expension Wall Insulation	1485	S.F.	1000	1.49	0.25	1.1581875	$ 430	1-Carp	0.09	1.9955	$ 267				$ 697	07 21 13.10 0040	212
29	Expansion Windows	20	Ea.	9	2.22	272	1.1581875	$ 6,301	1-Carp	39	1.9955	$ 1,556				$ 7,857	08 52 13.10 2050	274

30	Counter Partition	122.6666667	sq ft	45	2.73	19.35	1.52969925	$ 3,631	3-Carp	70.5	1.7264	$ 14,930				$ 18,561	10 57 23.19 1300	369
31	Conference Room Desk	3	Ea.													$ 2,625		
32	Attendant Desk	2	Ea.	16	0.13	2375	1.22767875	$ 5,831	1-Carp	22	1.9955	$ 88				$ 5,919	12 56 51.10 0100	418
33	Conference Room/Breakout Room Chair	30	Ea.													$ 2,430		
34	Couches	5														$ 3,150		
35	Coffee Table																	
36	Meeting Space Benches	120	L.F.	40	3.00	83	1.1581875	$ 11,536	2-Carp	35.3	1.9955	$ 8,453				$ 19,988	12 67 13.13 1500	420
37	Roof Deck Insulation	5520	S.F.	2000	2.76	0.26	1.166583	$ 1,674	2 Rofc	0.6	1.7771	$ 5,886				$ 7,560	07 22 16.10 0020	215
38	Roof Firproofing	5520	S.F.	3000	1.84	0.79	1.15390275	$ 5,032	G-2	0.71	1.9071	$ 7,474	0.1	1.467	$ 810	$ 13,316	07 81 16.10 0500	240
39	Roof 5-ply Gravel	18	ton	117	0.15	24	1.1223	$ 485	B-34B	2.41	1.7563	$ 76	5	1.351	$ 122	$ 683	03 05 13.25 0500	51
40	Suspended Ceiling	5520	S.F.	1875	2.94	1.66	1.1827215	$ 10,838	3 Carp	2.13	2.0137	$ 23,676				$ 34,514	09 51 23.30 1110	317
41	Closet Shelving	50	SF Shlf	175	0.29	4.61	1.0407375	$ 240	1 Sswk	1.2	1.4001	$ 84				$ 324		
42	Reception Desk Area	1	Station	1	1.00	1750	1.4431125	$ 2,525	2-Carp	705	1.7264	$ 1,217				$ 3,743	11 17 13.10 3000	381
43	Interior Walls Painting	8820	S.F.	2700	3.27	0.06	1.20692925	$ 639	2-Pord	0.46	1.8395	$ 7,463				$ 8,102	09 91 23.72 0240	342
44	Exterior Walls Painting	4470	S.F.	1625	2.75	0.27	1.20692925	$ 1,457	2-Pord	0.38	1.8395	$ 3,125				$ 4,581	09 91 13.60 1400	335
45	Demolish ADA Ramp	14.81481481	C.F.	33	0.45				B-9	66	1.21	$ 1,183	115.5	1.21	$ 2,070	$ 3,254		
46	Modify existing ADA Ramp	25	L.F.	12.22	2.05	305	1.354	$ 10,328	C-14H	143	1.4183	$ 5,070	1.88	1.091	$ 51	$ 15,450	03 30 53.40 4520	73
47	W8x10 for Angled Roof	170	L.F	600	0.28	13.75	0.957	$ 2,237	E-2	4.5	1.400	$ 1,071	2.49	1.077	$ 456	$ 3,764	05 12. 23.75 0300	130
48	W24x84	160	L.F.	1080	0.15	116	0.957	$ 17,762	E-5	3.61	1.400	$ 809	1.5	1.077	$ 258	$ 18,829		
49	Ceiling Suspension Assembly	5520	S.F.	1300	4.25	0.76	1.183	$ 4,962	2 Carp	0.88	2.014	$ 9,782				$ 14,744		
															Total Cost	$ 399,884		

			Means 2013 Pg. 718		Means 13	Means 2013 Pg. 735		
ADJUSTMENT FACTORS	Waste	Tax	Mat. City Index	Material WTC	Labor Overhead	Inst. City Index	Labor FC	Equip. C
Metal Decking (Roof)	1	1.0875	0.957	1.041	1.300	1.077	1.400	1.077
Steel Truss Joists	1.05	1.0875	0.957	1.093	1.300	1.077	1.400	1.077
Demolish Non-Structural Walls	1	1.0875	1.358	1.477	1.300	1.002	1.303	1.002
Steel Trusses Material	1	1.0875	0.957	0.957	1.300	1.077	1.400	1.077
Steel Trusses Installation								
Demolish Structural Colums	1	1.0875	1.171	1.273	1.3	1.21	1.21	1.21
Demolish Roof	1	1.0875	1.171	1.273	1.3	1.21	1.21	1.21
Mobilize - Crane, up to 75 ton	1	1.0875		0.000	1.3	1.091	1.41830	1.091
Mobilize - Forklift	1	1.0875		0.000	1.3	1.091	1.41830	1.091
Mobilize - Hyraulic Pump	1	1.0875		0.000	1.3	1.091	1.41830	1.091
Reshoring	1.06	1.0875	1.098	1.266	1.3	1.317	1.71210	1.317
Column Foundation Excavation	1	1.0875	1.358	1.000	1.3	1.002	1.30260	1.002
Column Formwork	1.06	1.0875	1.1	1.268	1.3	1.479	1.9227	1.479
Concrete Column	1.06	1.0875	1.175	1.354	1.3	1.331	1.7303	1.331
Concrete - footings	1.06	1.0875	1.098	1.266	1.3	1.317	1.7121	1.317
Column Foundation Formwork	1.06	1.0875	1.1	1.268	1.3	1.479	1.9227	1.479
Interior Walls	1.03	1.0875	0.975	1.092121875	1.3	1.549	2.0137	1.549
Expansion Wood Flooring	1.03	1.0875	1.014	1.13580675	1.3	1.268	1.6484	1.268
Interior Carpet	1.06	1.0875	1.014	1.1688885	1.3	1.268	1.6484	1.268
Conference Room Door	1	1.0875	1.065	1.1581875	1.3	1.535	1.9955	1.535
Expansion Exit Door	1	1.0875	0.957	1.0407375	1.3	1.077	1.4001	1.077
Expansion Exterior Walls	1.06	1.0875	0.975	1.12393125	1.3	1.549	2.0137	1.549
Expansion Windows	1	1.0875	1.065	1.1581875	1.3	1.535	1.9955	1.535
Expansion Insulation	1.06	1.0875	1.012	1.166583	1.3	1.367	1.7771	1.367
Attendant Desk	1.06	1.0875	1.065	1.22767875	1.3	1.535	1.9955	1.535
Meeting Space Benches	1	1.0875	1.065	1.1581875	1.3	1.535	1.9955	1.535
Roof Deck Insulation	1.06	1.0875	1.012	1.166583	1.3	1.367	1.7771	1.367
Roof Fireproofing	1.06	1.0875	1.001	1.15390275	1.3	1.467	1.9071	1.467
Roof 5-ply Gravel	1	1.0875	1.032	1.1223	1.3	1.351	1.7563	1.351
Suspended Ceiling	1.06	1.0875	1.026	1.1827215	1.3	1.549	2.0137	1.549
Counter Partition	1.06	1.0875	1.327	1.52969925	1.3	1.328	1.7264	1.328
Shelving	1	1.0875	0.957	1.0407375	1.3	1.077	1.4001	1.077
Desk Reception	1	1.0875	1.327	1.4431125	1.3	1.328	1.7264	1.328
Interior Walls Paint	1.06	1.0875	1.047	1.20692925	1.3	1.415	1.8395	1.415

Cost Estimate For Interior Elements (Square Foot Costs 2015)					
Element	**Unit**	**Unit Cost**	**Cost per S.F**	**S.F. of buildin**	**Est. Cost**
HVAC	S.F. Floor	$29.89	$29.89	5520	$164,992.80
Electrical Service/Distribution	S.F. Floor	$3.54	$3.54	5520	$19,540.80
Lighting and Branch Wiring	S.F. Floor	$8.56	$8.56	5520	$47,251.20
Communications and Security	S.F. Floor	$1.98	$1.98	5520	$10,929.60
Other Electrical Systems	S.F. Floor	$0.22	$0.22	5520	$1,214.40
Sprinkles	S.F. Floor	$5.92	$5.92	5520	$32,678.40
Standpipes	S.F. Floor	$1.72	$1.72	5520	$9,494.40
Total					$286,101.60

Direct Cost	$685,985.40
Indirect Cost	$137,197.08
Total Cost	$823,182.48
Markup	$41,159.12
Price	$864,341.60

Appendix K

Cost Estimate quantity hand calculations

Activity 1: Metal Deck Roof

Unit: SF

$L = 80$ ft

$W = 69$ ft

$A = 80' \times 69'$

$= 5520\ ft^2$

Activity 2: Steel truss joists

Unit: linear ft

L of 1 joist: 69 ft

# of joists = 9

$LF = 9 \times 69'$

$= 621$ ft

Activity 3: Steel Truss Material

Unit: ton

Length of 1 angled member: 11.20'

# of angled member: $18 \times 2 = 36$

Wt/ft of LL $3\frac{1}{2} \times 3\frac{1}{2} \times \frac{5}{16} \times \frac{3}{8}$ = 12.2 plf

Weight of LL = $11 \times 36 \times 12.2$ plf

= 4920 lb

Length of 1 HSS member: 80

# of HSS member = 4

Wt/ft of HSS = 55.53 plf

Weight of total HSS = $4 \times 80' \times 55$ plf

= 17600 lb

Total weight = 17600 + 4920 = 22520 lb

$22520\ lb \times \dfrac{1\ ton}{2000\ lb} = 11.26$

Total weight of steel material = 11.26

Activity 6: Demolish structural columns

Unit: C.F

Height of 1 column = 15'

L = 2 ft

W = 2 ft

C.F of 1 column = 15' x 2' x 2' = 60 ft^3

# of columns = 4

Total C.F. of column = 240 ft^3

Activity 7: Demolish Roof

Unit: C.F.

Length of roof = 65 ft

Width of roof = 69 ft

Depth of roof = 8 in

Volume of roof = 65' x 69' x $\frac{8"}{12"}$ = 7475 ft^3

Activity 14: structural wall Reshoring

Unit: S.F

W = 65 ft

H = 15 ft

L = 69 ft

Total SF = (65'x15' + 69'x15') 2

= 4020 SF

Activity 15: column Foundation Excavation

Unit: BCY

L = 4.5' → excavation = 5'

W = 4.5' → excavation = 5'

D = 3' → excavation = 3'

# of footings excavated = 4

CF = 4.5 ft x 4.5 ft x 3 ft x 4 = 350 ft^3

BCY = 350 ft^3 x $\frac{1\ yd^3}{27\ ft^3}$ = 13.0 CY

Activity 16: Column Formwork

UNIT: SFCA

Height of column = 15 ft

Diameter of column = 2 ft

perimeter = π (2 ft) = 6.28 ft

# of columns = 4

SFCA = 15' x 2' x 6.28 x 4

= 480 ft^2

Activity 17: concrete columns

Unit: CY

Height = 15 ft

Diameter = 2 ft

# of columns = 4

Volume = 15' x π/4 (2 ft)^2 x 4

= 188.49 ft^3 x yd^3/27 ft^3

Volume = 7.00 C.Y.

Activity 18: column Foundation formwork

Unit: SFCA

Height = 3 ft

Length = 4.5 ft

Width = 4.5 ft

footings = 4

column area = 2 ft^2 x π/4

= 3.14 ft^2

columns = 4

SFCA = (3' x 4.5' x 4.5' x 4) - 4 (3.14)

= 216 ft^2

Activities 19 - 20: concrete for Footing

Unit = CY

H = 3'

L = 4.5'

W = 4.5

= 4

volume = 3' x 4.5' x 4.5 x 4

= 243 ft^3 x yd/27 ft^3

volume = 9 cy

Activity 21: Interior Walls (Installation)

unit: S.F. H: 15'

L: 11.5' x 2, 11' x 2, 19' x 1, 9' x 4, 5.5' x 1, 5' x 2, 6' x 1,

15 x (23' + 22' + 19' + 36' + 5.5' + 10' + 6')

= 1822.5 S.F.

Activity 22: Expansion Wood Flooring

unit: S.F. L: 69' W: 15'

69' x 15' = 1035 S.F.

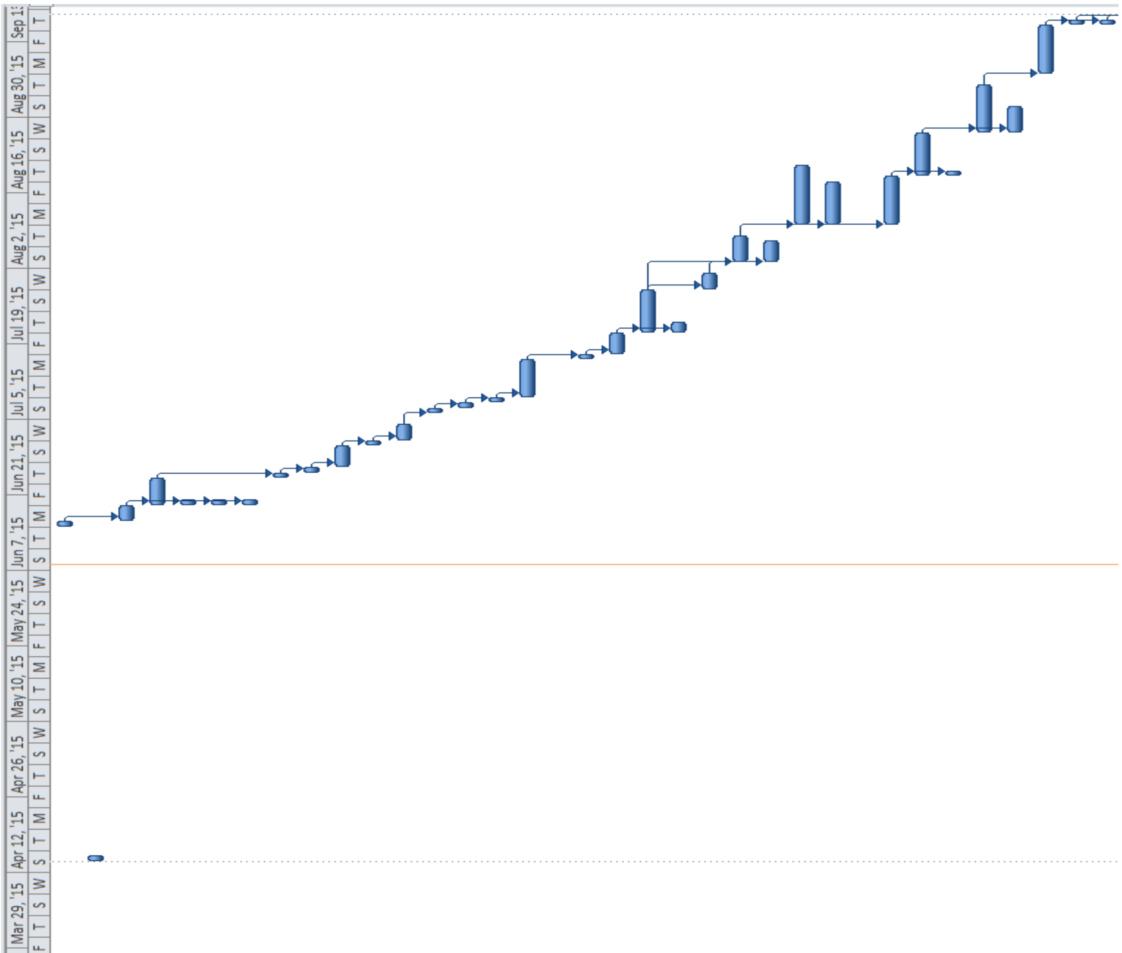
Activity 23: Interior Carpeting

unit: S.Y. L: 65' W: 69'

(65'/3) x (69'/3) = 500 S.Y.

Appendix L

Construction Schedule Trial 1, with an end date past the summer deadline.



Task Mode	Task Name	Duration	Start	Finish
	Mobilization	1 day	Mon 6/15/15	Mon 6/15/15
	Modify Existing Remaining Ramp	1 day	Tue 4/14/15	Tue 4/14/15
	Structural Wall Temporary Shoring	3 days	Tue 6/16/15	Thu 6/18/15
	Modify Remaining ADA Ramp	3 days?	Fri 6/19/15	Tue 6/23/15
	Demolition of Roof	1 day	Fri 6/19/15	Fri 6/19/15
	Demolition of Interior Columns	1 day	Fri 6/19/15	Fri 6/19/15
	Demolition of Non-Structural Walls	1 day	Fri 6/19/15	Fri 6/19/15
	Demolition of ADA Ramp	1 day	Wed 6/24/15	Wed 6/24/15
	Column Foundation Excavation	1 day	Thu 6/25/15	Thu 6/25/15
	Column Foundation Formwork	2 days	Fri 6/26/15	Mon 6/29/15
	Column Foundation Installation	1 day	Tue 6/30/15	Tue 6/30/15
	Column Formwork	3 days	Wed 7/1/15	Fri 7/3/15
	Column Installation	1 day	Mon 7/6/15	Mon 7/6/15
	Steel Truss Installation	1 day	Tue 7/7/15	Tue 7/7/15
	Steel Truss Joist Installaion	1 day	Wed 7/8/15	Wed 7/8/15
	Installation of Expansion Exterior Walls	5 days	Thu 7/9/15	Wed 7/15/15
	Expansion Exterior Walls Insulation	1 day?	Thu 7/16/15	Thu 7/16/15
	Roof Metal Deck Installation	2 days	Fri 7/17/15	Mon 7/20/15
	Roof Deck Insulation Installation	6 days	Tue 7/21/15	Tue 7/28/15
	Roof Fireproofing	2 days	Tue 7/21/15	Wed 7/22/15
	Top Expansion Exterior Walls	3 days	Wed 7/29/15	Fri 7/31/15
		5 days	Mon 8/3/15	Fri 8/7/15
	Electrical Rough Ends	4 days	Mon 8/3/15	Thu 8/6/15
	Suspended Ceiling	9 days	Mon 8/10/15	Thu 8/20/15
	Installation of Interior Non-Structural Walls	6 days	Mon 8/10/15	Mon 8/17/15
	Counter Partition	7 days	Mon 8/10/15	Tue 8/18/15
	Meeting space benches	6 days	Wed 8/19/15	Wed 8/26/15
	Desk Reception Installation	1 day	Wed 8/19/15	Wed 8/19/15
	Interior Walls Paint	7 days	Thu 8/27/15	Fri 9/4/15
	Exterior Paint	3 days	Thu 8/27/15	Mon 8/31/15
	Carpet	7 days	Mon 9/7/15	Tue 9/15/15
	Install Doors	1 day	Wed 9/16/15	Wed 9/16/15
	Move in furniture	1 day	Wed 9/16/15	Wed 9/16/15